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Updated rock mechanical design criteria for the Liikavaara Östra open pit

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Abstract

The assessment of rock mechanical data is a fundamental step in the process of evaluating the profitability of an open pit mine. The steepness of bench, interramp and resulting overall slope angles have a direct effect on the stripping ratio, therefore on the profitability of the mining operation. With the recently challenging metal prices in mind, the maximization of slope angles is even more important nowadays, prompting detailed investigations in the topic.

The primary goal of this master thesis is to provide updated rock mechanical design criteria for the Liikavaara Östra open pit in Northern Sweden, by collecting and assessing the already available information from the area, as well as evaluating the raw data of the drilling campaign undertaken in 2016.

After the compilation of the rock mechanical and geological data of the site, the bench scale slope stability was assessed with probabilistic and deterministic approach. During the process, two failure criteria (Mohr-Coulomb and Barton-Bandis) and two groundwater conditions (drained and undrained) were tested. Based on the gained experience in the neighboring Aitik mine, the drained Barton-Bandis scenario was used both in probabilistic and deterministic approach to recommend bench face angles. Based on the findings of the bench slope analysis, an increase of the bench and interramp angles (compared to the previous design study) is possible, with the presumption of improved smooth blasting techniques and minimized back break. In identified areas with excessively poor rock quality, the application of external rock support was assessed in order to maintain reasonable bench angles and avoid mining of additional waste rock. The application of external slope support practices deemed feasible in the footwall domains of the pit, where with the utilization of support methods, the possible relocation of the E10 road can be avoided. The overall slope stability was also assessed in the footwall with limit equilibrium analysis methods. The footwall of the pit was found to be stable with the new design criteria, although it is sensitive to the presence of groundwater pressures.

Keywords Slope stability, kinematic analysis, probabilistic analysis, deterministic analysis, external rock support, Liikavaara Östra

Foreword

First and foremost I want to thank everyone who helped me in any way during my university studies and the writing of my master thesis. This thesis is the result of many year process and without the help of countless people along the way I could not have been successful. Thank you all very much once again! This thesis work was funded by Boliden Mineral AB, which hereby is acknowledged.

I want to thank my supervisor Professor Mikael Rinne from Aalto University, who sparked my interest in rock mechanics and supervised this thesis. I am particularly grateful to Per-Ivar Marklund, former Head of the Rock Mechanics Department at Boliden, who made it possible that I could write my thesis at the company, moreover guided me at the start of my project.

Many thanks to my thesis advisor Adjunct Professor Jonny Sjöberg (Itasca Consultants AB, Sweden), whose experience and knowledge in open pit slope stability was invaluable. His insights and suggestions significantly improved not only this thesis but my knowledge in rock mechanics and slope stability as well. LKAB and Anton Bergman deserves a special gratification; without them, the point load tests could not have been conducted. Furthermore, Anton's help during the design process and personal discussions were utmost helpful. I am also grateful to Sofia Höglund and Mats Larsson, who helped me during my data collection trip in Aitik.

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Abbreviations

BFA	Bench Face Angle
BRQD	Boliden Rock Quality Designation
EE	East End domain code
FoS	Factor of Safety
FW	Footwall domain code
GSI	Geological Strength Index
HW	Hanging Wall domain code
IRA	Interramp Angle
ISRM	International Society for Rock Mechanics
JCS	Joint Compressive Strength
JRC	Joint Roughness Coefficient
KLSZ	Kiruna Ladoga Shear Zone
LKAB	Luossavaara-Kiirunavaara Aktiebolag
LOMP	Life of Mine Plan
NDZ	Nautanen Deformation Zone
NE	Northeast domain code
OSA	Overall Slope Angle
PLT	Point Load Test
PoF	Probability of Failure
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SEK	Swedish Kronor
UCS	Unconfined (or Uniaxial) compressive strength of rock
WE	West End domain code

Symbols

a	Axial UCS Value
d	Diametric UCS Value
D	Disturbance Factor
Ei	Intact Rock Modulus
I_S	Point Load Index
$I_{s(50)}$	Size Corrected Point Load Index
mi	Intact Material Constant
MR	Modulus Ratio
α	Bench Angle
δ	Dip of Toppling Bedding Planes
σ_H	Largest Horizontal Stress
σ_h	Minimum Horizontal Stress
σ_v	Vertical Stress
ϕ	Friction Angle
ψ_f	Dip of Face
ψi	Dip of Intersection of Two Failure Planes
ψ_p	Dip of Failure Plane
ψ_s	Dip of Surface

1. Introduction

1.1. Objective, scope and outline of the thesis

Liikavaara Östra mine in Northern Sweden is a planned satellite pit of the Aitik mine, owned and operated by Boliden Mineral AB. The last rock mechanical assessment of the planned mine is dated from 2008. As the project moved forward from conceptual stage to prefeasibility stage, the update of the current design criteria and detailed assessment of rock mechanical data became necessary in order to satisfy the needs of further project development. The goal of this master thesis is to give updated, more detailed rock mechanical design parameters for Liikavaara Östra.

1.1.1. Objective

The purpose of this thesis work is to compile and analyze the existing rock mechanical data from the Liikavaara area, and with the newly acquired information on rock characteristics (rock strength, RMR value, etc.) and joint properties (orientation, filling) develop new rock mechanical design criteria for the planned open pit. Beside the rock mechanical properties these parameters are established with respect to the geology and structures of the rock mass, while considering the mining method induced back break and blast damage.

After the compilation of old and new data, optimal bench and interramp slope angles are recommended, which are based on the kinematic, probabilistic and deterministic analysis of the different domains. Furthermore, empirical catch bench width calculations are conducted as well. The overall slope stability of the pit is also revised where required.

In order to focus the research, several questions are defined to which this study aims to find the suitable and satisfactory answers:

- How does the new information on rock conditions modify the current design criteria?
- What are the differences in rock conditions and structures between Aitik and Liikavaara Östra?
- What are the recommendations based on the rock conditions for pit design criteria?
- Are there specific areas where, due to poor rock conditions, rock treatment/reinforcement is necessary?
- What type of rock treatment/reinforcement methods are advised for the slope situated in poor rock?

1.1.2. Scope

While the following attributes are also important for a sufficient analysis of slope stability, due to the size of the project these are out of the scope of this thesis:

- Groundwater levels of the slopes and hydrogeological assessment of the area (although alternative scenarios were studied for the stability of the overall slope angles)
- The effect of haul roads to slope stability
- Numerical modeling of slopes
- Detailed modeling of rockfall catch benches

1.1.3. Outline

The following steps are taken to achieve the aims of the project:

- Compilation and initial assessment of existing data:
  Assessing the already obtained rock mechanical, geological and structural data
- Collection of data:
  Broadening the information base of the area through collecting rock mass and joint properties
- Determination of design domains:
  Design domains are established with respect to pit geometry, geology, structures, and the joint attributes
- Bench slope stability analysis of the design domains:
  Kinematic analysis of design sectors with *DIPS 7.0* (Rocscience, 2016b), further analysis with the *ROCPLANE 3.0* (Rocscience, 2016c), *SWEDGE 6.0* (Rocscience, 2016d), and *ROCTOPPLE 1.0* (Rocscience, 2016e) software tools using probabilistic and deterministic approaches
- Calculations for external rock support need:
  In specific areas, the external rock support need is also calculated
- Overall slope stability check:
  Analysis of selected design sections with *SLIDE 7.0* (Rocscience, 2016a) limit equilibrium analysis tool.
- Establishment of final design criteria:
  By summarizing the project the recommended bench and inter-ramp slope parameters are presented
- Conclusions and recommendation for further research:
  Concluding the thesis work and proposing areas for further work

1.2. Overview of Liikavaara Östra

The northern part of Norrbotten County is an important mining district of Sweden. Historically, mining of iron ore started in the 18th century and since then the industry is a major contributor to the economy of the county. The major mines in the area are the Kiruna and the Malmberget underground iron ore operations of LKAB and the Aitik open pit copper mine of Boliden Mineral AB. Beside these mines, multiple other orebodies were mined in the past, are in production today, or planned for future exploitation.

Figure 1 Location of the Aitik operation (ezilon.com, 2016).

The Aitik mine is a large scale open pit operation 20 km east of Gällivare, which exploits a heavily metamorphosed and modified porphyry Cu-Au-Ag deposit (Wanhainen, 2005). The host rocks of the mineralization are muscovite schist, biotite gneiss and amphibole-biotite gneiss (Boliden, 2014). Aitik was discovered in the 1930s, and mining started in 1968 with 2 Mt per year ore production. Since then the rate of extraction has been increased and in 2015, the annual ore production was 36 Mt. With further expansion of production, this rate will grow to 45 Mt by the year of 2017 (Boliden, 2016a). Also, some 45-50 Mt of waste rock will be extracted each year. The ore grades and tonnages of Aitik and Liikavaara Östra deposits are displayed in Table 1.

The ore is mined with large scale production blasting, then mucked and hauled to the in-pit crusher with a high capacity truck fleet. For improved slope stability presplitting and scaling are done where required. The ore is crushed and conveyed further to the processing plant, where milling, flotation, and thickening takes place.

The ore concentrate is transported further to Boliden Rönnskär smelter via train, while the tailings are stored in the tailings pond next to the processing plant west from the mine. Waste rock from the pit is also transported by the truck fleet to the waste rock dumps which are located around the main pit (Figure 2).

Figure 2 Overview of Aitik (location of the Liikavaara Östra pit is marked with red) (Lantmäteriet, 2016).

Table 1 Mineral Reserve and Resource estimates for the Aitik, Salmijärvi, Aitik Östra and Liikavaara Östra (Boliden, 2014).

Deposit 31.12.2013	**Classification**	**Quantity Mt**	**Au g/t**	**Ag g/t**	**Cu %**	**Mo g/t**
Aitik	Proven reserve	691	0.15	1.6	0.22	24
	Probable reserve	228	0.14	1.1	0.21	27
	Measured resource	207	0.10	1.1	0.16	16
	Indicated resource	1132	0.10	0.9	0.17	26
	Inferred resource	185	0.11	0.4	0.13	23
Sälmijärvi	Proven reserve	71	0.09	1.0	0.19	28
	Probable reserve	13	0.09	0.9	0.20	29
	Measured resource	42	0.05	0.8	0.15	26
	Indicated resource	175	0.05	0.8	0.15	24
	Inferred resource	5	0.04	0.7	0.14	29
Aitik Östra	Proven reserve	-	-	-	-	-
	Probable reserve	37	0.08	0.4	0.15	16
	Measured resource	-	-	-	-	-
	Indicated resource	117	0.10	0.3	0.13	14
	Inferred resource	36	0.09	0.3	0.11	13
Liikavaara Östra	Proven reserve	-	-	-	-	-
	Probable reserve	45	0.07	2.7	0.28	43
	Measured resource	-	-	-	-	-
	Indicated resource	42	0.06	2.1	0.24	28
	Inferred resource	-	-	-	-	-

The planned Liikavaara Östra Cu-Au open pit mine will function as a satellite pit of the Aitik mine and is located 3 km east of the main pit (Figure 2 and Figure 3). The mining is currently planned to start in 2023, with a total life of mine of 5 years. The ore production of Liikavaara Östra is planned to be between 7-12 Mt and the total output (ore and waste rock) between 16 and 18 Mt per year. The current tonnage and grade of the mineral resources and reserves are shown in Table 1. Note that the satellite pit is currently in pre-feasibility stage, thus these numbers are likely to change.

As mentioned previously the Liikavaara Östra mineralization is situated east from the main orebody of Aitik. The ore zone is orientated to west and has a steep (in some places sub-vertical) dip (average of 80°). The host rocks are low grade metamorphosed sedimentary rocks, while the ore is situated in foliated biotite schist. The main ore minerals are chalcopyrite, pyrite as well as pyrrhotite. A detailed discussion of regional and local geology can be found in Chapter 2 (Boliden, 2014). In Figure 4 the relative locations of Aitik-Salmijärvi and Liikavaara Östra ore bodies are displayed.

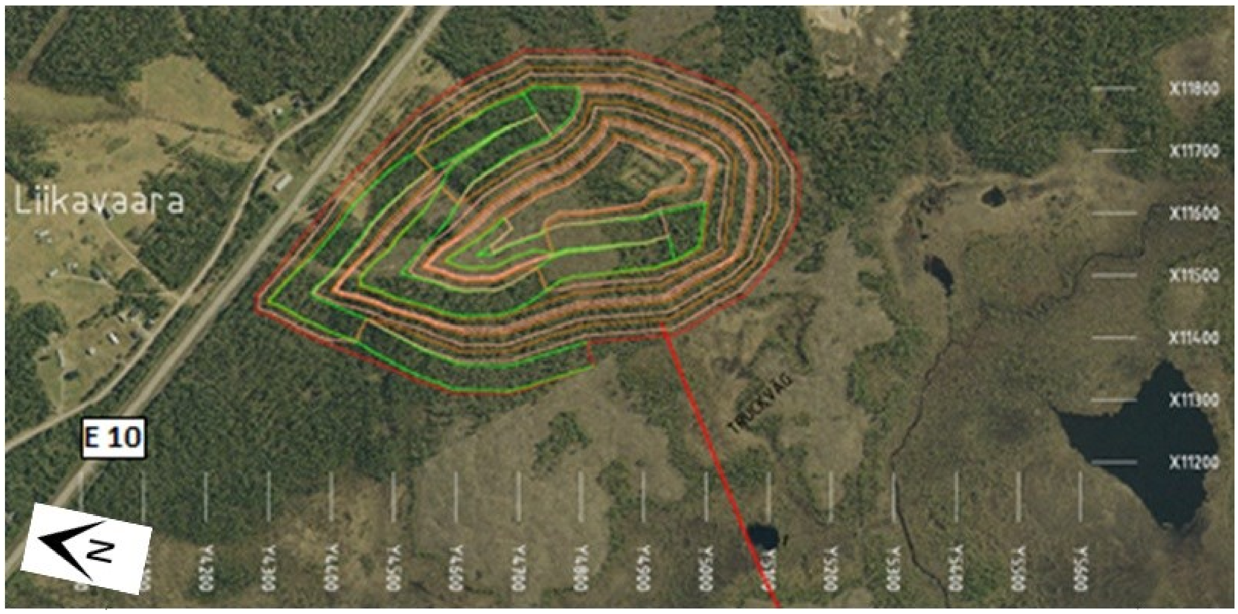

Figure 3 The planned pit of Liikavaara Östra (Wiik, 2010).

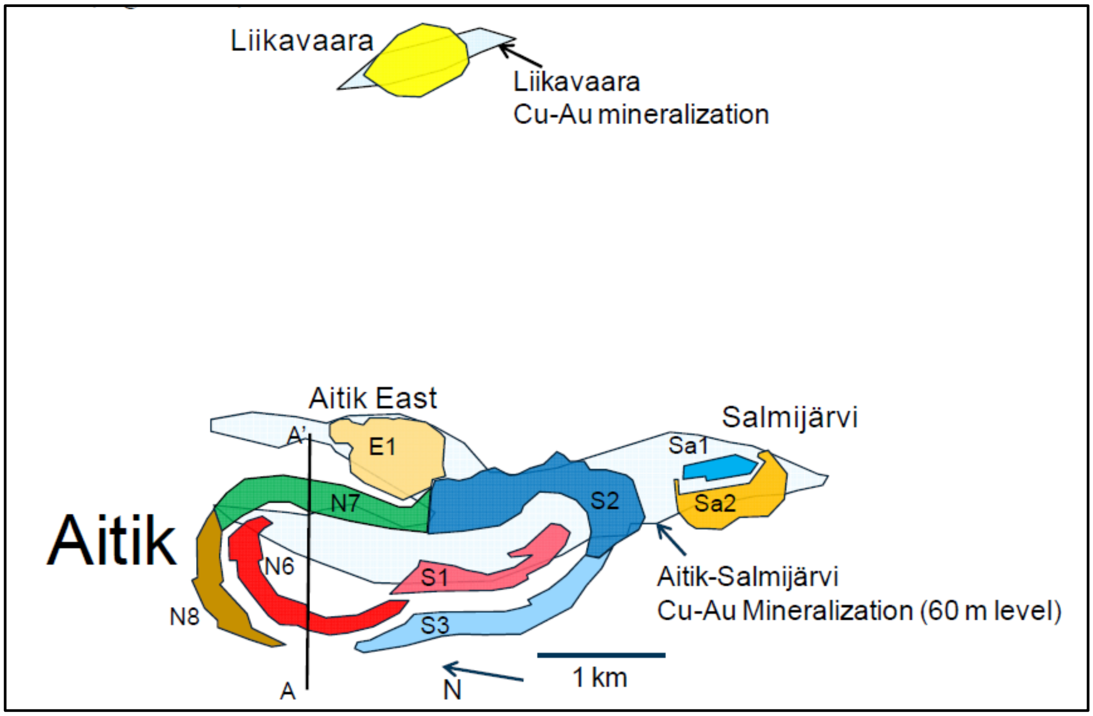

Figure 4 Relative location of Liikavaara Östra to the Aitik-Salmijärvi orebody (Boliden, 2014).

It must be noted here that multiple coordinate systems had been used during the life of the Aitik mine. At present day, all systems use the Mine North ("Gruvnorr") referred as just "North" in Aitik and Liikavaara Östra. Apart from the general orientations in the Introduction chapter, this paper refers to Mine North the same way as well, unless stated otherwise. In Figure 5 the different coordinate systems are displayed as compared to each other.

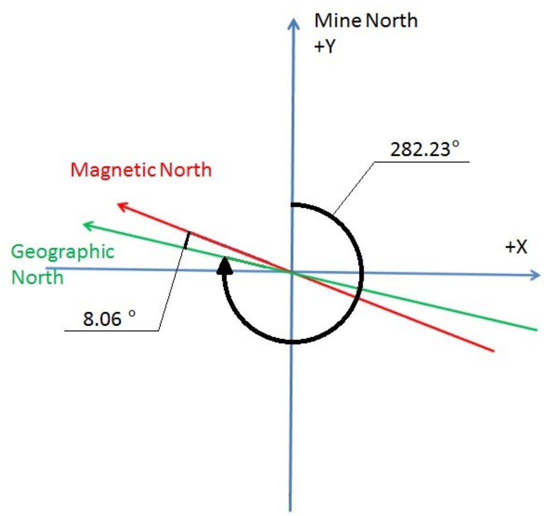

Figure 5 Mine North compared to Magnetic and Geographic North.

2. Geology

2.1. Regional geology

The Liikavaara Östra deposit is situated on the Baltic shield, which is an exposed Precambrian part of the Eastern European Craton. The deposit lies along the Kiruna Ladoga Shear Zone (KLSZ), a major shear zone environment of two major provinces of the shield, the Svecofennian Province and the Karelian Province, see Figure 6 (Monro, 1988).

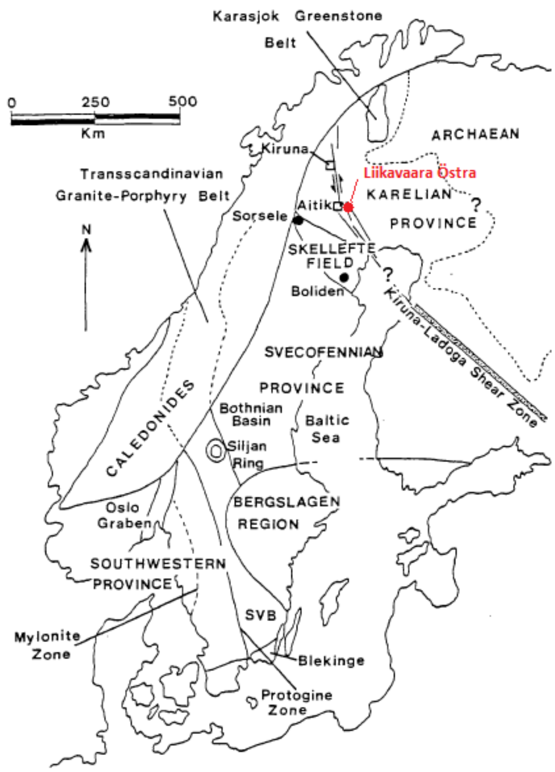

Figure 6 Overview of the Baltic Shield (Monro, 1988).

The basement of Northern Norrbotten consists of Archean granites and gneisses dated to 2.8 Ga old, (Wanhainen, et al., 2005; Monro, 1988). This is overlaid by formations from the Paleoproterozoic Svecokarelian orogeny (1.96 and 1.75 Ga ago). In the stratigraphy first the Karelian rift-related rocks occur from 2.5 to 2.0 Ga. Above the Karelian, 1.9 Ga old Svecofennian shallow marine supracrustals are deposited (Wanhainen, et al., 2005). These metavolcanic and metasedimentary units later are intruded by multiple suites, first the Haparanda (1.89-1.86 Ga) and the Perthite-monzonite (1.88-1.86 Ga) suites. The Lina granite intruded 1.81-1.78 Ga ago (Bergman, et al., 2001), see Figure 1-1 in Appendix 1. The Liikavaara Östra deposit is located in this Svecofennian metasedimentary (supracrustal) environment, with diorites (Haparanda suite) and granites (Lina granite suite) in the vicinity (Figure 7) (Bergman, et al., 2001).

2.2. Local setting and deposit geology

Liikavaara Östra is an epigenetic Cu-Au deposit situated in a regional deformation zone called the Nautanen Deformation Zone (NDZ), see Figure 7 and Figure 1-1 in Appendix 1. In this (geographic) NNW–SSE trending, 3 km wide zone several mineral deposits are located (Bergman, et al., 2001). The ore zone of Liikavaara Östra is located on the (geographic) eastern leg of an overturned synform structure. Host rock environment consists of volcanic (andesite) and sedimentary rocks (conglomerate and turbidite) and intrusions of several rock types (aplite, diorite, gabbro, and granite). The deposit has a steep dip between 85° and 75° and has a dip direction to the (geographic) west (Figure 8 and Figure 9) (Wiik, 2010).

The current resource and reserve estimates calculate with 45 Mt of probable reserves at 0.28% Cu, 0.07 g/t Au, 28 g/t Mo and 2.7 g/t Ag. The indicated resource model contains 42 Mt at 0.24% Cu, 0.06 g/t Au, 43 g/t Mo and 2.1 g/t Ag, see Table 1 (Boliden, 2014).

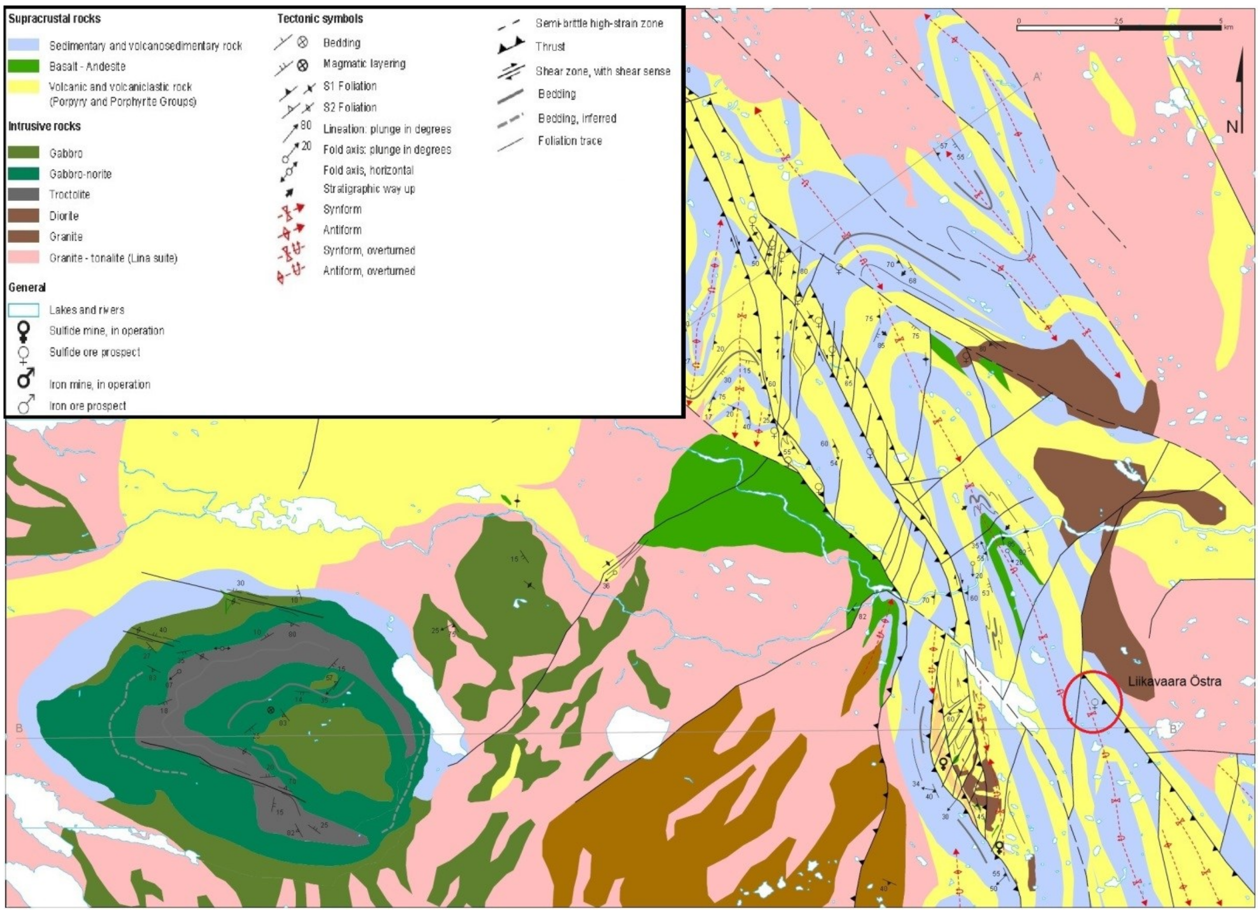

Figure 7 Geology of the Aitik- Malmberget area (Referenced to Geographic North) (Drejing-Carroll et al., 2016).

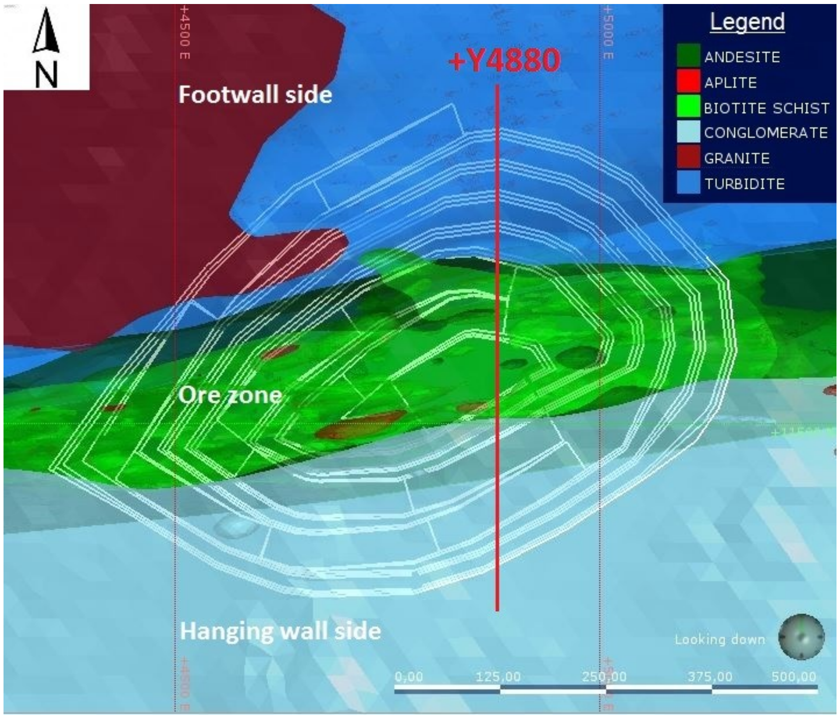

Figure 8 Plan view of host rocks and mineralization at Liikavaara Östra, referenced to Mine North (Höglund, 2016a). The indicated cross section is displayed in Figure 9.

2.2.1. Hanging wall units

The hanging wall of the orebody consists of conglomerates and other sedimentary rocks similar, which are for simplification classified as conglomerates in the geological nomenclature of Boliden (Figure 8). Original bedding and large fragments in these rocks can be observed as no, or only low-grade metamorphism was identified in the hanging wall units (Zweifel, 1976). Minor anisotropy is present in the rock mass, which is due to the sedimentary origin. The contact between the conglomerate and the orebody is gradual (Wiik, 2010).

2.2.2. Ore zone units

The ore mineralization in Liikavaara Östra occurs in biotite schist which is the result of the metasomatic conversion of andesite. Towards the footwall side unaltered and slightly altered andesite bodies are more present. Within the mineralization aplite intrusions are also observed, see Figure 8 and Figure 9. The ore-forming alteration is not exclusive to the biotite schist, in the footwall rocks pyrite mineralization occurs, which decreases further away from the biotite schist (Wiik, 2010). The biotite schist is also partially altered to chlorite and has a distinct foliation (Monro, 1988; Zweifel, 1976).

The primary ore mineral in Liikavaara Östra is chalcopyrite; moreover, pyrite and pyrrhotite are also present. The mineralization occurs in disseminated form. Although the Pb and Zn grades are considered low in the deposit (below 1%), galena and sphalerite are common minerals in the ore. Scheelite likewise occurs throughout the deposit. The most frequent gangue minerals are quartz and calcite (Monro, 1988; Zweifel, 1976). During the 2007 drill program W, Sn minerals were described with Re as an occurrence in the tungsten oxide. There is molybdenite in the ore as well, usually with higher concentrations in the aplite bodies and quartz veins. The distribution of ore minerals changes along the orebody, as the mineralization is zoned. On both sides of the deposit, pyrite dominates which turns into chalcopyrite in the middle of the orebody (Wiik, 2010).

2.2.3. Footwall units

In the footwall of the ore, multiple rock types are present. The majority of the footwall consists of low-grade metamorphosed turbidite, with some presence of andesite. There is also a smaller dioritic intrusion in the eastern end of the planned pit. Close to the surface in of some drill holes conglomerate can be identified as well, specifically in the north-eastern section of the area. In the western end of the pit, Lina granite is present, where some sections possess a so-called "vuggy" texture and foliation. Although the Lina granite dominates the surroundings, there is no direct contact between the ore and the granite (Figure 8) (Wiik, 2010).

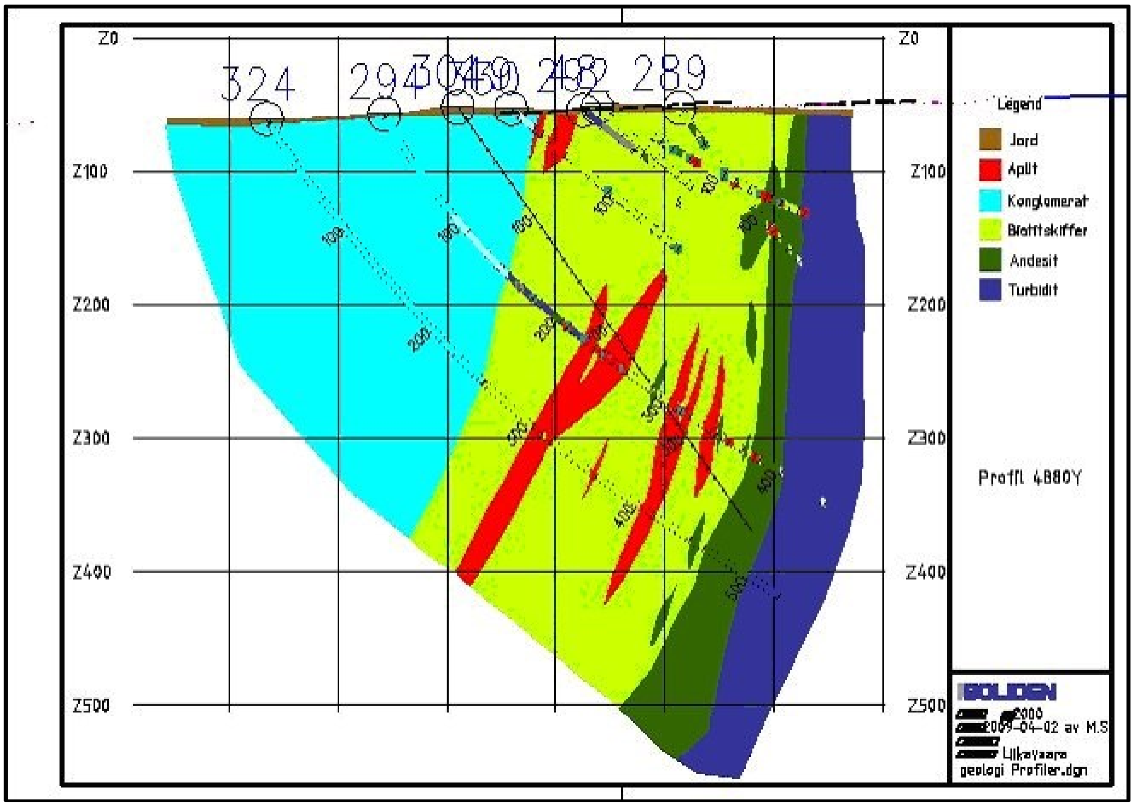

Figure 9 Cross section of the mineralization at profile +Y 4880 at Liikavaara Östra (Wiik, 2010). The ore mineralization is connected to biotite schist (light green), aplite (red) and andesite (dark green).

2.3. Structural geology

From Archean to Quaternary the district has been undergone multiple ductile and brittle deformations. Moreover fold structures are recognized in Norbotten, and the relation between fold and deformation structures is distinct in the Gällivare area (Bergman, et al., 2001). As mentioned earlier, The Liikavaara Östra deposit is situated in the NDZ, which is a 3 km wide ductile shear zone in the Gällivare area with the trend of NNW–SSE. Strong ductile deformations occur along the discontinuity zone, which has sub-vertical to vertical dip (Bergman, et al., 2001).

Large parts of other shear zones in Norbotten, such as the Pajala and Karesuando-Arjeplog deformation zones (see Figure 1-1 in Appendix 1) reactivated as brittle deformation zones (Bergman, et al., 2001). Based on geophysical measurements in the Liikavaara area and the significant core loss in some of the drill holes, it can be assumed that the NDZ reactivated as brittle deformation zone similarly to the adjacent shear zones. The fold structure in the Liikavaara area is interpreted as an overturned syncline parallel to the direction of the NDZ. The syncline is NNE-SSW directed and steeply dipping between 75° to 85° (Figure 10) (Bergman, et al., 2001).

Units of Liikavaara Östra are low/medium grade metamorphosed and in general have a lower metamorphic grade than the formations in Aitik. This is due to the fact that the NDZ is a boundary between metamorphic zones in the area. The foliation of the host rocks and the ore zone are parallel to the units indicating that the metamorphosis occurred before the folding events (Bergman, et al., 2001). In general, it can be stated that foliation and anisotropy have a higher presence in the footwall units than in the hanging wall rocks. Strong foliation (schistosity) occurs in the ore zone. From 1.75 Ga to present day major erosion took place and eroded at least 5 km of rock to expose mineralization on the surface like Aitik and Liikavaara Östra at present day (Monro, 1988), see Figure 10.

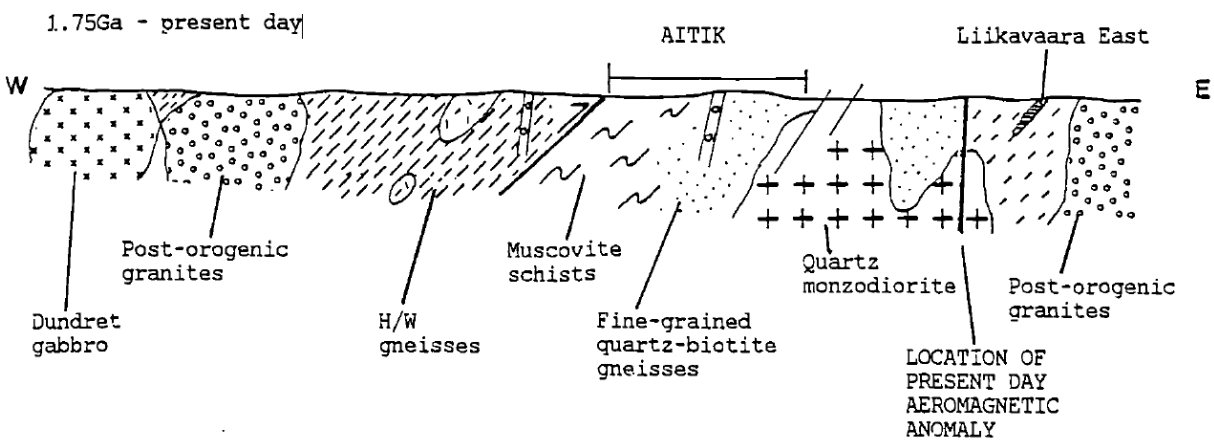

Figure 10 Tectonic setting of the Gällivare area (Monro, 1988).

As it was mentioned previously, the mineralization lies along the NDZ, and significant structures and zones of discontinuity are observed around the orebody. In 2013 a geophysical study was conducted by GeoVista AB to detect and further define the position of the deformation zones (Mattson & Thunehed, 2013). In this study 15 possible zones were identified with 3D resistivity and refraction seismic methods. These areas are characterized by low seismic velocity and RQD values which verify the brittle features of these structures. Except for one structure, all zones are trending geographic NW-SE or NNW-SSE and dipping vertically or sub vertically. One particular discontinuity differs from this trend as it is dipping with 30° to the geographic north. The width of these zones is mostly between 5 and 10 m, although one zone along the orebody footwall contact stands out with its 30 to 60 m width. The length of the structures varies between 100 and 800 m (Mattson & Thunehed, 2013). In Figure 11 the discontinuity zones are displayed along the 3D geological model of Liikavaara Östra (Höglund, 2016a).

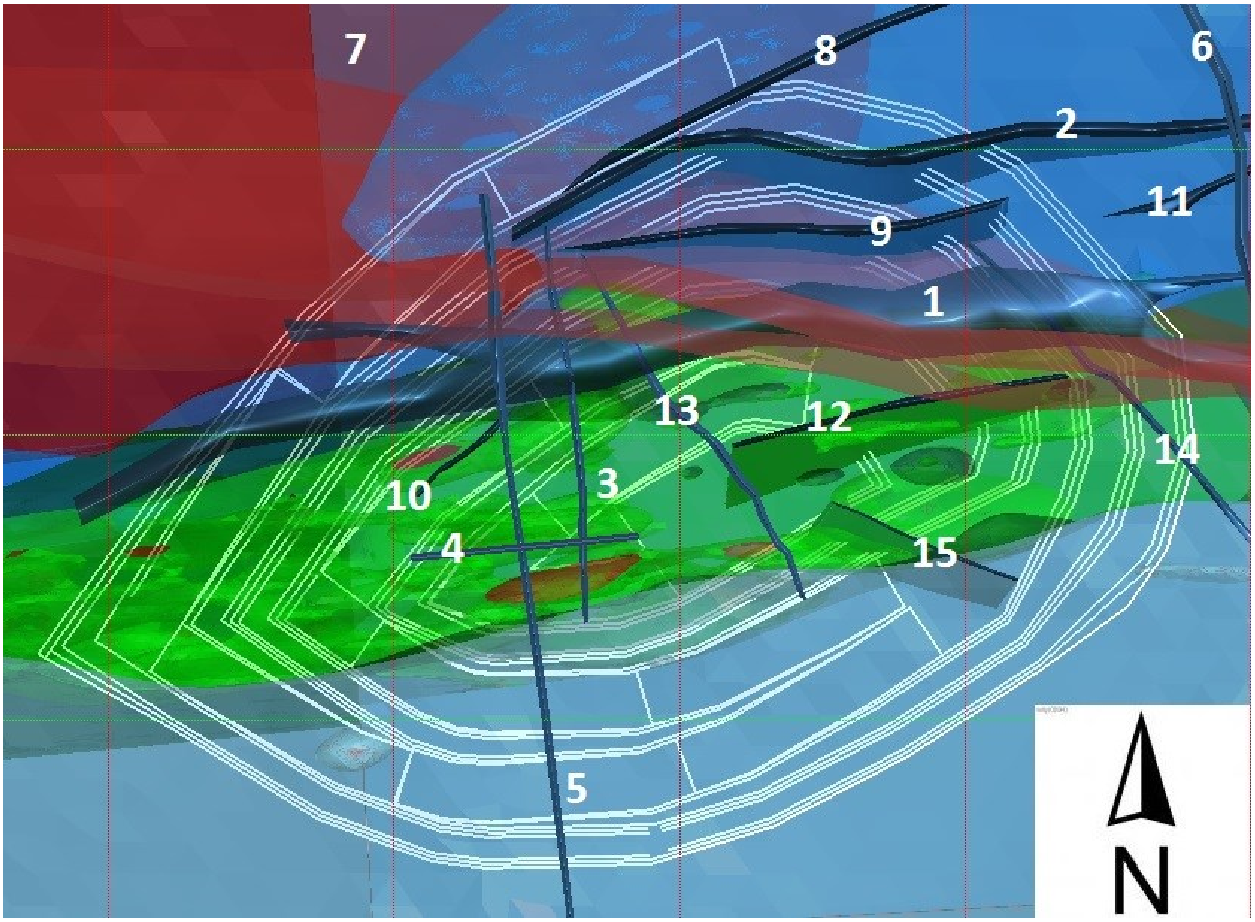

Figure 11 Interpreted deformation zones in Liikavaara Östra after (Höglund, 2016a; Mattson & Thunehed, 2013), figure referenced to Mine North. The numbers refer to the codes used in the report of (Mattson & Thunehed, 2013).

2.4. Hydrogeology

In the area of Liikavaara Östra, no hydrogeological studies have been conducted until the writing of this thesis. Preliminary hydrogeological assessment is planned to be carried out later during 2016. Limited information regarding the presence of water can be retrieved from the condition of the geological and rock mechanic drill holes. In all holes the water level is close to or at the collar. As the source of the water can be either the glacial till or the bedrock, broad conclusions cannot be drawn. For this reason, both bench and overall scale stability were checked with both undrained and drained hydrogeological conditions assumed, and the final design recommendations are based on drained benches and partially depressurized overall slope. The assumption of depressurized mining benches and partially dry slopes are based on the observation in the Aitik pit and previous studies (Sjöberg, et al. 2016), but needs to be verified in the next project stage.

3. Rock mechanics conditions

In this chapter, the overall rock mechanical conditions of Aitik, Salmijärvi, and Liikavaara Östra areas are assessed, more precisely the rock mass and joint properties of the Aitik and Salmijärvi pits. The data of Liikavaara Östra in this section is briefly compared with the above mentioned mines. The detailed compilation and analysis of the 2016 drilling campaign executed in Liikavaara Östra are presented in Chapter 4.

3.1. Stress conditions

The regional stress conditions in Northern Scandinavia are controlled by movements of tectonic plates, and the maximum horizontal stress has an approximate (geographic) E-W to NE-SW direction, although it varies significantly in certain areas, see Figure 12 (Sjöberg, et al. 2016; Heidbach, et al., 2008).

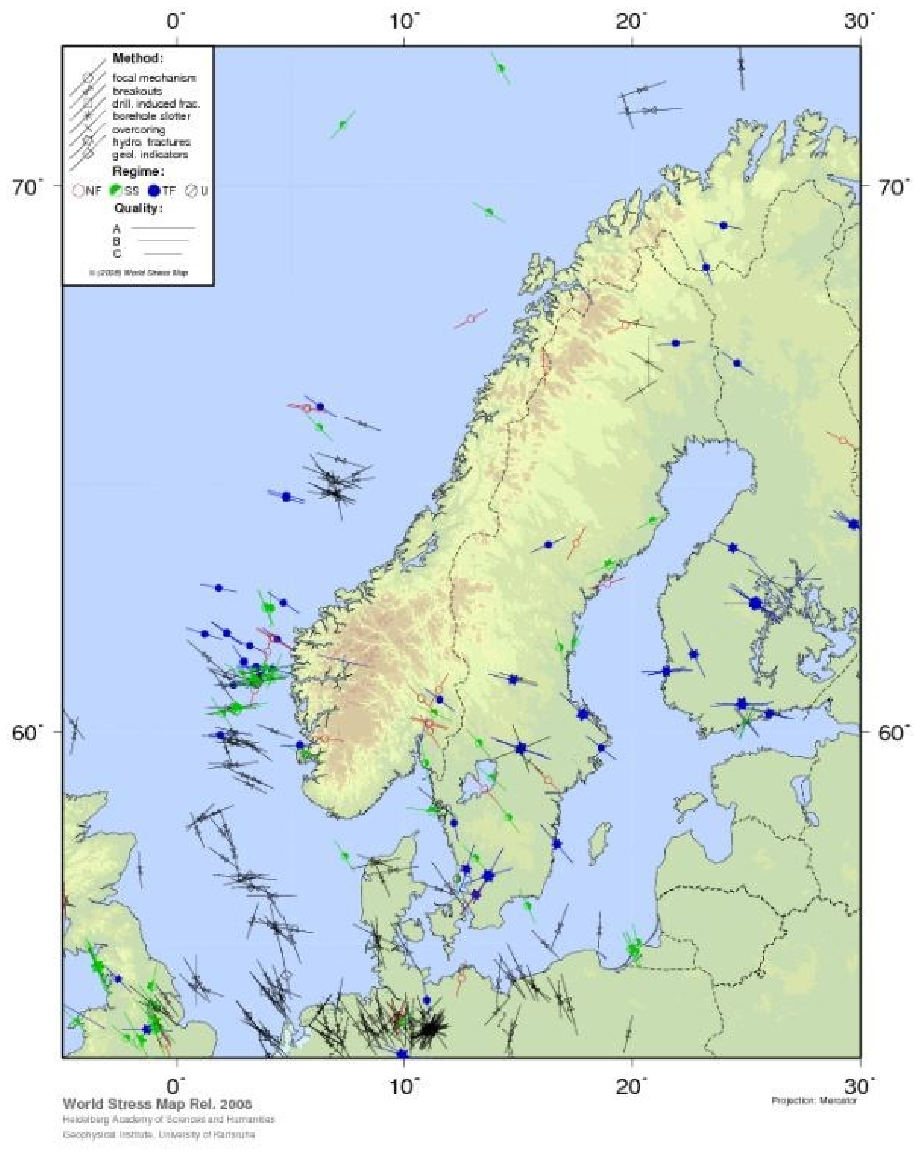

Figure 12 Orientation of principal stresses in Scandinavia (Heidbach, et al., 2008).

The virgin state of stress has not been measured in Aitik nor Liikavaara Östra by the time of writing this thesis. Due to the lack of measurements, extrapolated values of stress state are used. In case of Aitik and Liikavaara Östra, the closest place of analysis took place in the Malmberget mine of LKAB, 20 km (geographic) NW from the planned satellite pit. In Malmberget several stress field measurements and stress calibrations have been executed. Calibrations were conducted with numerical modeling by Sjöberg (2008) and Perman et al. (2016) as cited in (Sjöberg, et al. 2016). The stress relations calculated for the Malmberget mine have been quantified as follows (Sjöberg, et al. 2016):

- $\sigma_H = 0.0396z$ [MPa]
- $\sigma_h = 0.0161z$ [MPa]
- $\sigma_v = 0.0270z$ [MPa]

Where σ_H denotes the maximum horizontal stress, σ_h is the minimum horizontal stress, and σ_v is the vertical stress. These values were assumed to be valid for Aitik. The directions of the maximum and minimum horizontal stresses are the following (Sjöberg, et al. 2016):

- 122° from (geographic) North for σ_H.
- 32° from (geographic) North for σ_h

The direction of these local horizontal stresses roughly align with the regional stress regime (NW-SE) indicated on Figure 12. This location of stress directions concurs with the thrust faulting identified in the area (Heidbach, et al., 2008; Wanhainen, 2005).

But if the stress orientations are compared to the local geology in Liikavaara Östra (see Figure 13), it can be seen the horizontal stress directions are not matching the general observation, that the major horizontal stress aligns with the dip direction of the orebody, and the minor horizontal stress is parallel to the strike. As there have been no local measurements at Liikavaara Östra it cannot be verified that either the stresses are following the regional regime (and the directions determined in Malmberget are valid) or the directions have been locally re-orientated along the orebody. However, due to the direction and magnitude of the projected stresses and the size of the pit (200 m planned final depth), it can be reasonably assumed that the virgin rock stress will not significantly influence the slope stability in Liikavaara Östra (Sjöberg, 2016).

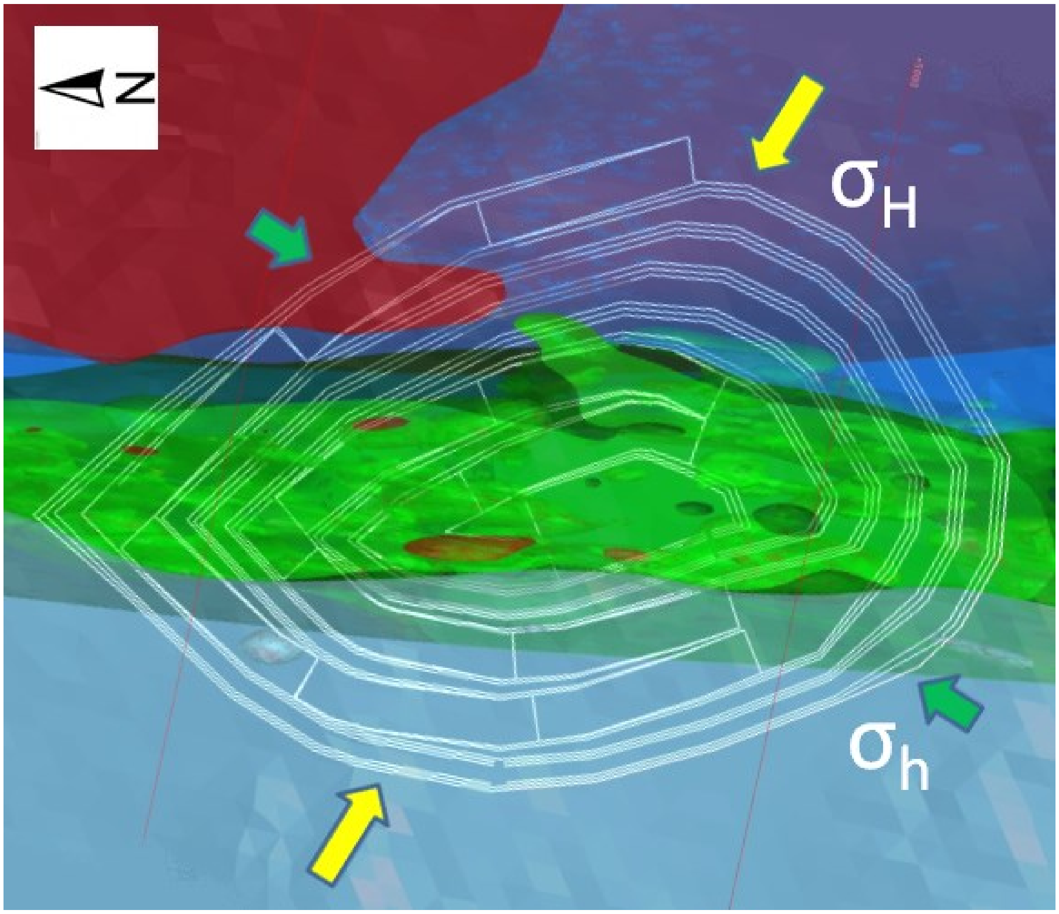

Figure 13 Orientation of horizontal stresses around the Liikavaara Östra pit (Referenced to Geographic North). After Sjöberg et al. (2016) and Höglund (2016b).

3.2. Rock mechanical properties

The Uniaxial Compressive Strength (UCS) values are obtained from Point Load Tests (PLT), the Rock Mass Rating (RMR) is based on the Boliden standard Rock Quality Designation (BRQD, described in 3.2.2) and joint characteristics acquired through rock mechanical core logging. The data sets of Liikavaara Östra were gathered in the internal reports of Krauland & Romedahl (1996) and Bergman (2008a). In the main pit multiple rock mechanical studies have been conducted regarding rock mass properties and structure orientations. The rock mass related data are derived from the most recent rock mechanical study, which is part of the 2016 Aitik Life of Mine Plan (LOMP) (Sjöberg, et al. 2016). The Salmijärvi related values are accessed from the 2008 rock mechanical report of Salmijärvi (Bergman, 2008b).

3.2.1. Intact rock strength

In Table 2 the UCS values of rock units in Liikavaara Östra, Aitik and Salmijärvi are compared. It can be seen that the intact rock strength of the footwall and hanging wall units are significantly higher in Liikavaara Östra, while the values of Aitik and Salmijärvi have roughly the same value range. On the contrary, the values of the ore zones are higher in Aitik and Salmijärvi, than in Liikavaara Östra. It must be noted that the standard deviation of the samples are greater in Liikavaara Östra, most probably due to the lower number of samples and the low-level accuracy of the point load test method.

Table 2 Comparison of UCS values between Aitik, Salmijärvi and Liikavaara Östra.

<table>
<tr><th colspan="2">Type of rock</th><th>Ore zone</th><th>Primary Host rocks</th><th>Secondary Host rocks</th></tr>
<tr><td rowspan="4">Mine</td><td rowspan="2">Liikavaara Östra</td><td>Biotite gneiss</td><td rowspan="2">Volcanic sediment</td><td>Andesite</td></tr>
<tr><td>Biotite schist</td><td>Epidote zone</td></tr>
<tr><td>Aitik</td><td>Biotite gneiss</td><td>Diorite, Leptite</td><td>Amphibolite</td></tr>
<tr><td>Salmijärvi</td><td>Biotite gneiss</td><td>Diorite</td><td>Amphibolite</td></tr>
<tr><td rowspan="5">Uniaxial Compressive Strength ([MPa], Average ± Std. Dev.)</td><td rowspan="2">Liikavaara Östra (Krauland & Romedahl, 1996)</td><td rowspan="2">77 ± 52</td><td rowspan="2">150 ± 58</td><td>157 ± 47</td></tr>
<tr><td>145 ± 88</td></tr>
<tr><td>Aitik (Sjöberg, 2005)</td><td>111 ± 81</td><td>93 ± 35</td><td>84 ± 20</td></tr>
<tr><td>Aitik (Sjöberg, et al., 2016)</td><td>87±49</td><td>112±35</td><td>87±30</td></tr>
<tr><td>Salmijärvi (Bergman, 2008b)</td><td>121±48</td><td>107±19</td><td>88±38</td></tr>
</table>

3.2.2. Rock mass characterization data

The quality of the rock mass is compared in RQD, BRQD, and RMR values. In Table 3 the standard and the Boliden RQD values are presented. Due to the differences in the standards, BRQD and RQD values must not be treated as equal.

Table 3 Comparison of RQD and BRQD values in Aitik, Salmijärvi and Liikavaara Östra.

Type of rock		Ore zone	Primary Host rocks	Secondary Host rocks
Mine	**Liikavaara Östra**	Biotite gneiss	Volcanic sediment	Andesite, Epidote zone
		Biotite schist		
	Aitik	Biotite gneiss	Diorite, Leptite	Amphibolite
	Aitik	Biotite gneiss, Biotite schist	Diorite	Amphibolite
	Salmijärvi	Biotite gneiss	Diorite	Amphibolite
RQD (Avg. ± Std. Dev.)	**Liikavaara Östra**	45 ± 28,	56 ± 24	67 ± 16
		52 ± 22		
BRQD (Average ± Std. Dev.)	**Liikavaara Östra**	N/A	37 ± 37	52 ± 35
		47 ± 36		
	Aitik (Marklund, 2006)	58	53	56
	Aitik (Sjöberg, et al., 2016)	62	58	66
		52		
	Salmijärvi (Bergman, 2008b)	69	39	55

The Boliden standard RQD (BRQD) is a modified version of the industry-wide used joint frequency assessment method, developed by Boliden Mineral AB. BRQD is a more sensitive tool, underestimating RQD values by approximately 30%. The method is differing from regular RQD in a way that it does not measure the lengths of samples longer than 10 cm over the length of core section; but counts the number of 5 or 10 cm long bits (depending on core diameter) that can be taken from the core over the length of the section examined (Boliden, 2000a).

In the comparison of the BRQD values, it is clear that the fracture frequency in Liikavaara Östra is lower (i.e., higher BRQD) than in Aitik and Salmijärvi. On the other hand, the RMR values from all sites (Table 4) show that the joint properties and overall rock mass quality are somewhat better in the footwall and hanging wall (primary and secondary host rocks) of Liikavaara Östra. Among the three ore zones Salmijärvi possesses the highest RMR values, while Aitik and Liikavaara Östra have roughly the same numbers.

Table 4 Comparison of RMR values in Aitik and Liikavaara Östra.

<table>
<tr><th colspan="3">Type of rock</th><th>Ore zone</th><th>Primary Host rocks</th><th>Secondary Host rocks</th></tr>
<tr><td colspan="2" rowspan="5">Mine</td><td rowspan="2">Liikavaara Östra</td><td>Biotite gneiss</td><td rowspan="2">Volcanic sediment</td><td rowspan="2">Andesite, Epidote zone</td></tr>
<tr><td>Biotite schist</td></tr>
<tr><td>Aitik</td><td>Biotite gneiss</td><td>Diorite, Leptite</td><td>Amphibolite</td></tr>
<tr><td>Aitik</td><td>Biotite gneiss, Biotite schist</td><td>Diorite</td><td>Amphibolite</td></tr>
<tr><td>Salmijärvi</td><td>Biotite gneiss</td><td>Diorite</td><td>Amphibolite</td></tr>
<tr><td rowspan="7">RMR</td><td rowspan="4">(Average ± Std. Dev.)</td><td rowspan="2">Liikavaara Östra</td><td>N/A,</td><td rowspan="2">67 ± 12</td><td rowspan="2">72 ± 12</td></tr>
<tr><td>70 ± 12</td></tr>
<tr><td rowspan="2">Aitik (Marklund, 2006)</td><td rowspan="2">64 ± 12</td><td>61 ± 14</td><td rowspan="2">63 ± 14</td></tr>
<tr><td>N/A</td></tr>
<tr><td rowspan="2">(Weighted mean)</td><td rowspan="2">Aitik (Sjöberg, et al., 2016)</td><td>70</td><td rowspan="2">69</td><td rowspan="2">71</td></tr>
<tr><td>64</td></tr>
<tr><td>(Average ± Std. Dev.)</td><td>Salmijärvi (Bergman, 2008b)</td><td>76 ± 13</td><td>60 ± 7</td><td>68 ± 11</td></tr>
</table>

From the comparison of the available UCS, BRQD and RMR values in Aitik, Salmijärvi, and Liikavaara Östra, it can be presumed that the overall rock quality is slightly worse in the planned satellite pit than in the mines already in production.

3.3. Joints and structures

In this section, the discontinuities of the Aitik and Salmijärvi pits are compared with the latest rock mechanical report of Liikavaara Östra (Bergman, 2008a). The results of Aitik are derived from the LOMP report (Sjöberg, et al. 2016) while the structural data of Salmijärvi is obtained from the master thesis of Perks (2015).

3.3.1. Discontinuities in Aitik

Large scale structures in Aitik

Although multiple geophysical, core logging, lineament studies, and structural mapping studies have been conducted in Aitik to identify large scale structures, no such discontinuities were successfully described apart from the hanging wall ore contact. The up to 30 m thick thrust fault is dipping into the slope of the hanging wall with an average of 46° (Figure 14). According to numerical models, toppling failures can develop around this structure at deeper mining depths (final pit depth 800 and 850 m), especially if the slope is not drained properly (Sjöberg, et al. 2016).

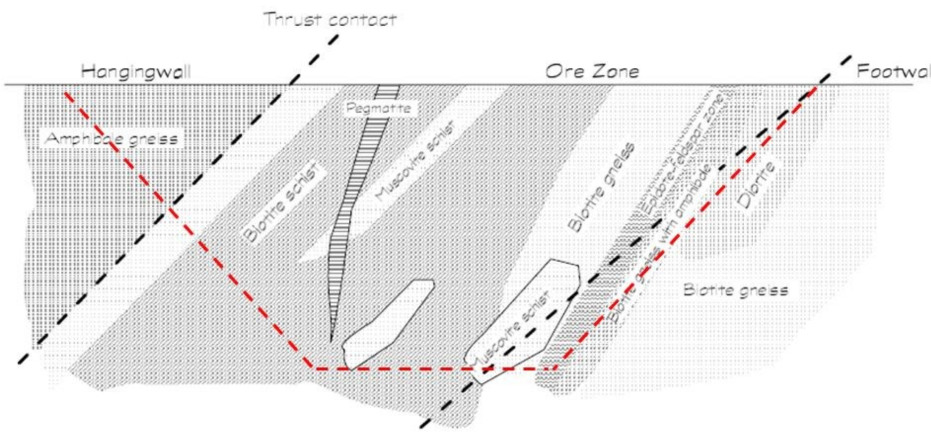

Figure 14 Simplified geological cross section of Aitik with sketched pit outline, modified after (Sjöberg, 1999).

Joints in Aitik

Multiple studies have been conducted to determine the orientation and properties of joints in Aitik. Cell mappings, line mappings, oriented core measurements, borehole video analysis and photogrammetric studies were performed by several authors and by Boliden staff to determine joint attributes. In the latest rock mechanics study (Sjöberg, et al. 2016) these previous studies were assessed, compared and design parameters were derived consequently. The joint sets based on the current database are shown in Appendix 2. In Figure 15 the sets are displayed by their corresponding design sectors in the pit.

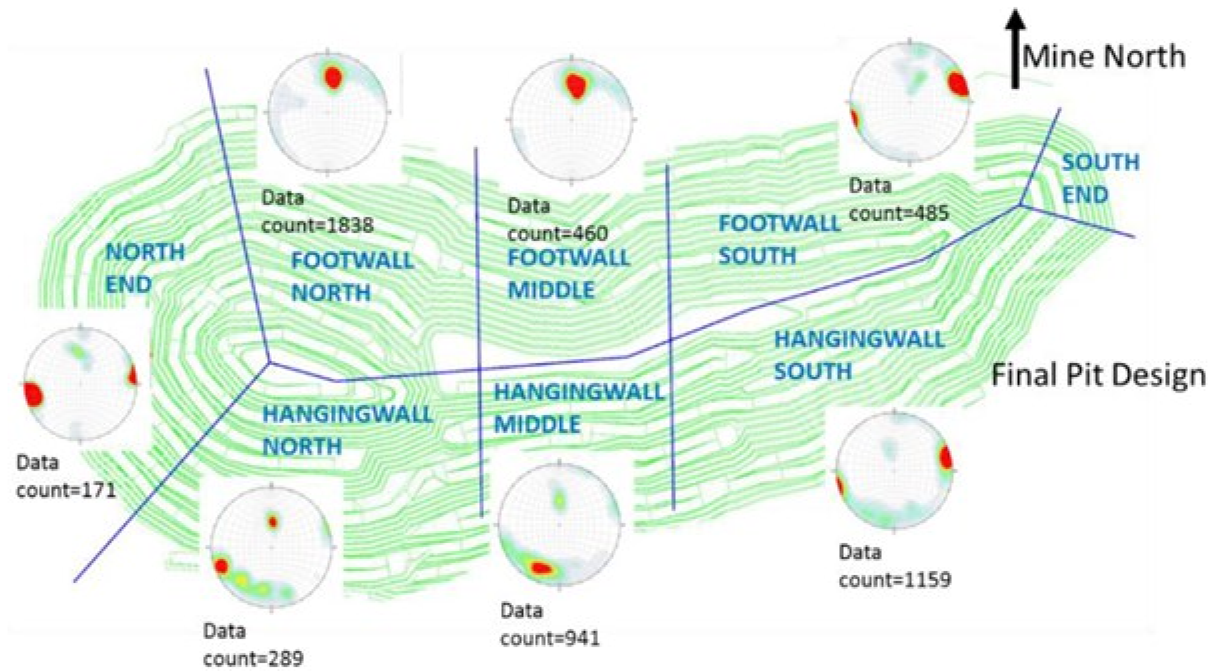

Figure 15 Joint orientations by design sectors in Aitik (Sjöberg, et al. 2016).

The most frequent joint sets in Aitik the high and middle angle foliation planes (JS1) which are dipping south parallel, sub-parallel to the ore body. Steeply dipping joints cutting across the ore zone (JS2 and JS3) are also dominant. The length of these joint sets are short (up to 10 m) with a small 1-2 m wide spacing. The sets are aligning with the large scale structural trends in the pit and an analogy between joints and structure trends can be presumed (Sjöberg, 1999).

Long joints can also be identified through the pit. Studies showed that these long discontinuities have a dip direction similar to the JS2 (subparallel to the orebody) and 45° to 65° dip. The length of these joints is varying between 15 and 120 m, with the shorter joints present in the footwall, while longer ones more current in the hanging wall. Joint spacing is 1 to 4 m. This orientation in the footwall is a source for potential planar failures (the joints “daylight” in the benches) which have been experienced multiple times, see Figure 16 and Figure 17. The presence of these structures is more frequent in the upper 180 m levels both in the footwall and the hanging wall (Sjöberg, 1999).

Figure 16 Long joints in the footwall (Sjöberg, 1999).

Figure 17 Planar failure of a bench in Aitik footwall, photo courtesy of Boliden.

3.3.2. Discontinuities in Salmijärvi

Large scale structures in Salmijärvi

In Salmijärvi possible large scale structures can be identified in borehole AITIK1211 and AITIK1219, where at given depth low BRQD and RMR values are present. In the first hole at 160 m the footwall contact can be found, while in the latter hole at 230-250 m the hanging wall contact can be determined. Neither of the possible structures considered having considerable effects on the overall slope stability of the pit (Perks, 2015).

Joints in Salmijärvi

Discontinuities were observed in Salmijärvi by borehole filming in 2008 and 2010 by Bergman and photogrammetry survey by Perks in 2015. In the rock mass three dominant sets were identified: (i) south dipping foliation planes, (ii) high angle east-west dipping set, and (iii) high angle north dipping system (Sjöberg, et al. 2016; Perks, 2015). The design sectors are based on these sets and displayed in Figure 18. The structure set data by design sectors are in Appendix 2.

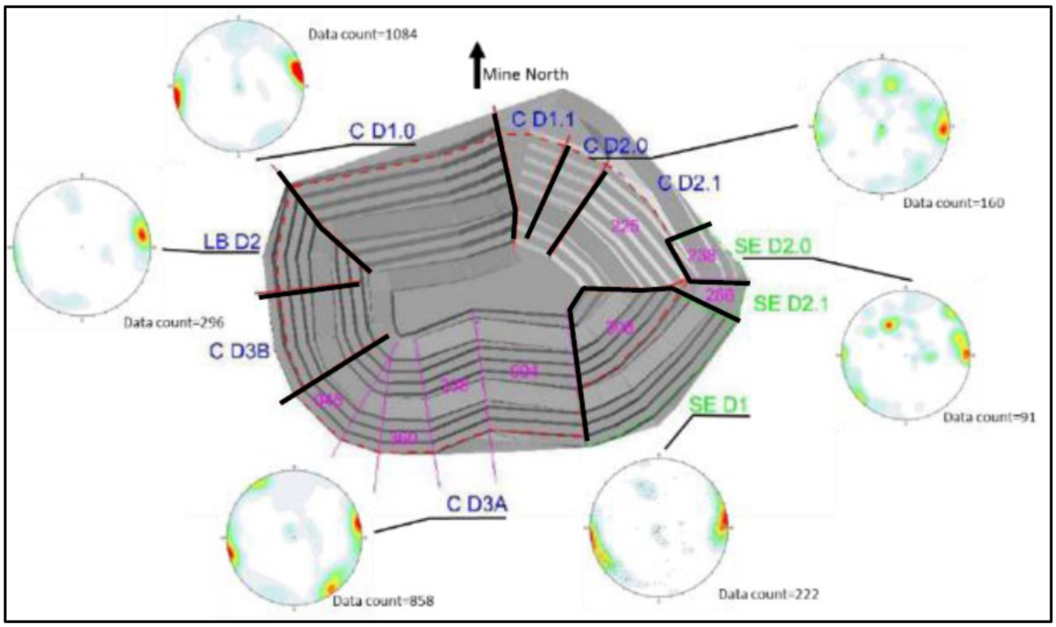

Figure 18 Salmijärvi pit design sectors with adherent joint sets (Perks, 2015).

3.3.3. Discontinuities in Liikavaara Östra

The results presented here are derived from the latest previous rock mechanic assessment of the pit, which is from 2008. In this study, the RQD and RMR data from the 1996 conceptual study were processed, and two more video inspected boreholes (AITIK331 and AITIK333) were evaluated. The available information dated from 2008 until present day are gathered and assessed in detail in Chapter 4.

Large scale structures in Liikavaara Östra

In Liikavaara Östra, the assumed large scale structures are the previously described crushed zones (see subchapter 2.3) mainly on the footwall side of the planned pit. These areas of discontinuities are known from drill holes and seismic surveying and assumed to be significantly affecting the stability of the Footwall pit area (Wiik, 2010). The zones are almost exclusively trending (geographic) NW-SE or NNW-SSE and have a vertical, sub-vertical dip, but with one zone trending to the (geographic) N with a 30° dip, see Figure 11 in chapter 2.3.

Joints in Liikavaara Östra

In the 2008 study, three major discontinuity sets have been identified in Liikavaara Östra, see Figure 19. Joint set 1 is parallel to the ore zone and similarly dipping; it was already indicated in the 1996 scoping study (Krauland & Romedahl, 1996). Joint set 2 has a lower dip angle than the ore zone and mostly occurs in the southern part of the area, while the dip direction is sub-parallel to the ore zone. Joint set 3 is practically a horizontal bedding set and only identified in the northern part of the planned pit (Bergman, 2008a).

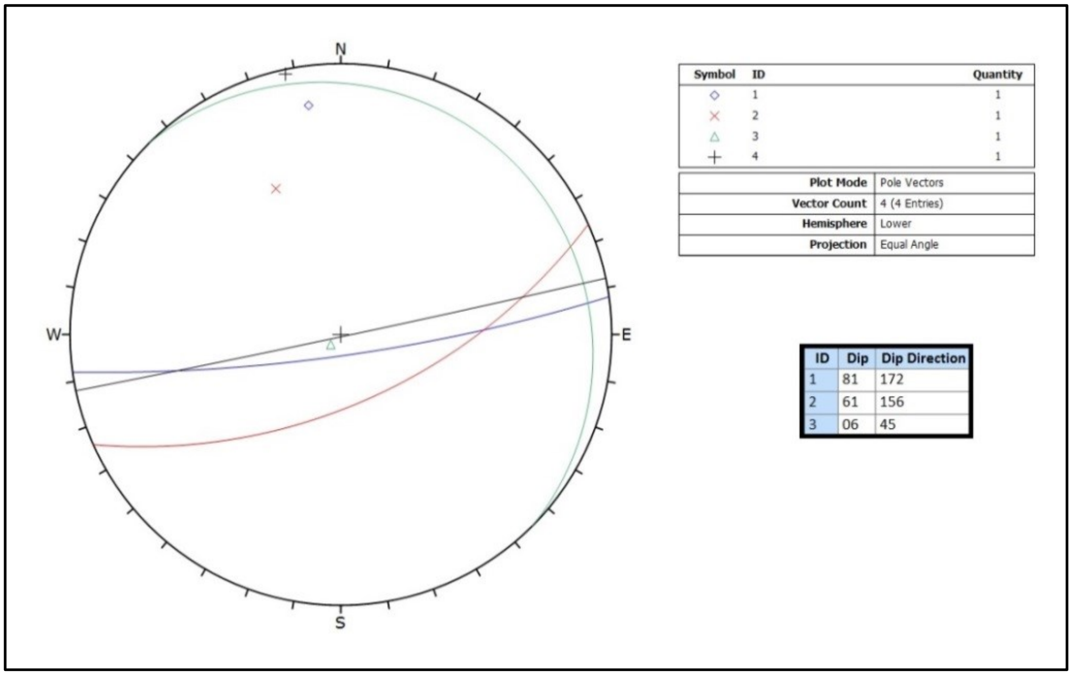

Figure 19 Stereographic projection of the major structures at Liikavaara Östra.
The 4th (black) set indicates the strike of the ore zone.

3.3.4. Comparison of joint sets in Aitik, Salmijärvi and Liikavaara Östra

Based on the abovementioned structural orientations a general comparison of the three pits can be made. In Figure 20, the sets of Aitik and Salmijärvi footwalls and hanging walls are displayed. For the joint sets of Liikavaara Östra, see Figure 19.

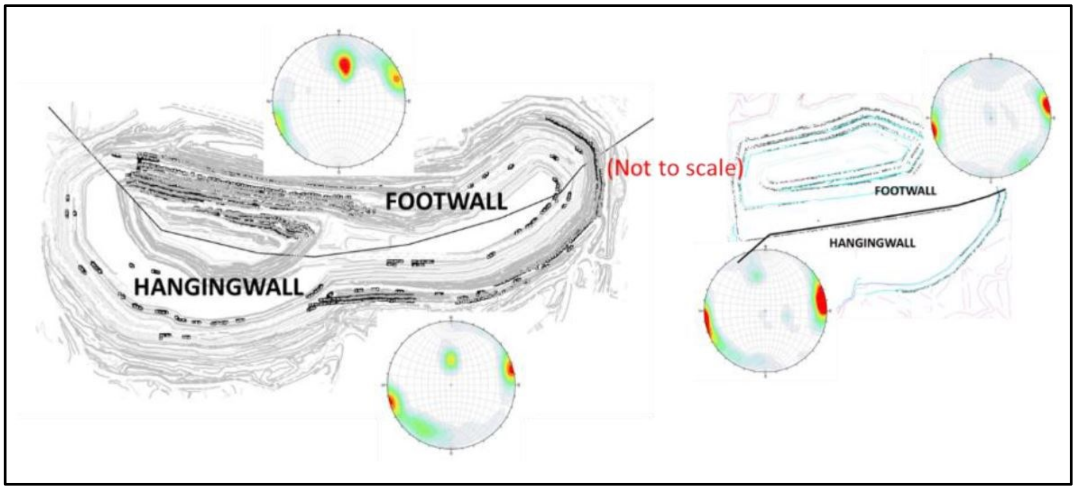

Figure 20 Joints sets of Aitik and Salmijärvi (Sjöberg, et al. 2016).

As the geology in Aitik and Salmijärvi is practically the same, the joint characteristics both in the hanging wall and the footwall are broadly matching. In the hanging wall of both pits, the NNW-SSE oriented system appears to be the most relevant structure. In the footwall sector of Aitik, the S dipping foliation is dominant, along with the NNE dipping set. Although the S dipping foliation is present in Salmijärvi with even steeper dipping, the NNE dipping system is the primary set (Sjöberg, et al. 2016).

Compared to these two pits, Liikavaara Östra shows similarities with the footwall section of Aitik. The two major joint sets of Liikavaara Östra are S dipping systems, while the third set is sub-horizontal bedding set, which does not appear to be dominant in either of the sectors in the already mined pits.

3.3.5. Joint shear strength properties

The shear strength of joints in Aitik has been investigated by several authors. Small scale test of sawn drill cores and shear tests on mated joint samples were performed by Call et al. (1976). In a later study, these values were re-evaluated by West et al. (1985) as cited in Sjöberg (1999). Simple field tilt tests were conducted by Sjöberg (1999) and the laboratory test results of previous studies were also assessed. Based on the results of the tilt tests and previous works, the cohesion values can be assumed as zero; which is also a conservative approach in the design process (Sjöberg, 1999). The results of Sjöberg (1999) are presented in Table 5.

As no joint shear strength data is available from Liikavaara Östra, the values obtained in the Aitik main pit were used for comparable rock types (biotite schist). For rock units without available friction angle values from Aitik, either values from other publications (for conglomerate and turbidite) or conservative assumptions were used as friction angle in the slope stability analysis for Liikavaara Östra (Table 6).

Table 5 Joint shear strengths in Aitik (Sjöberg, 1999).
In the last row, the Sjöberg et al. (2016) and West et al. (1985) names of the joint sets are presented.

Sector	Rock type	Joint set name	Friction angle (*=estimated)		Cohesion
			Typical value [°]	Approximate range [°]	
Hanging wall	Amphibole gneiss	All	28	27-30	0
Hanging wall	Hanging wall contact	All	25	20-30	0
Ore	Muscovite schist	All	30	30-32	0
Ore	Biotite schist	All	*30	*30-32	0
Ore	Biotite gneiss	All	32	30-34	0
Ore	Epidote-feldspar zone	All	*30	*30-34	0
Footwall	Amphibole-epidote-feldspar gneiss	All	*30	*30-34	0
Footwall	Diorite	All	32	30-35	0
Footwall	Biotite gneiss with amphibole	All	*32	*30-35	0
All	Long joints	HAE (JS2), HAW (JS5)	36	34-41	0

Table 6 Friction angles used in slope stability analysis in Liikavaara Östra.

Rock type	Friction angle [°]	Source
Biotite schist	30	Aitik Life of Mine Report (Sjöberg, et al. 2016)
Conglomerate	35	(Barton & Choubey, 1977)
Turbidite	30	Lack of data, conservative assumption
Granite	31	(Barton & Choubey, 1977)

3.4. Current slope designs in Aitik, Salmijärvi, and Liikavaara Östra

In all three open pits, the current slope designs are results of iterative design processes. The different slope angles and dimensions are calculated based on the quality of the rock mass (RMR, BRQD), joint orientations (dip and dip direction), joint properties (shear strength, roughness, filling etc.), available hydrogeological conditions (water level, pore pressure) and the conclusions originating from the previous relevant slope design. Although the bench designs are differing in the three mines, some dimensions are concurring due to the necessities of the mining machinery, production requirements, and rock fall retention criteria. These dimensions are displayed in Table 7.

Table 7 Design dimensions in Aitik, Salmijärvi and Liikavaara Östra.

	Aitik Footwall	Aitik Hanging wall	Salmijärvi	Liikavaara Östra Hanging wall	Liikavaara Östra Footwall
Single bench height	15 m	15 m	15 m	15 m	15 m
Double bench height	30 m	30 m	30 m	30 m	30 m
Catch bench width	11 m	11 m	11 m	11 m	11 m
Drilling offset	5.5 m	3 m	3 m	3 m	5.5 m

One of the exceptions is the drilling offset in the Footwall sectors in the Aitik pit. Here the drilling offset is lengthened to 5.5 m, so the same foliation plane is cutting the toes of both of the single benches (Marklund, et al. 2007). In Liikavaara Östra Footwall the offset is also planned to 5.5 m due to the expected low rock quality.

Maximum interramp heights and angles for the hanging wall and footwall in the main pit have been assessed through by numerical modeling in (Sjöberg, et al. 2016). The models were tested with 200 m interramp height, and the interramp slopes were calculated accordingly. This work supports the experience gained over the years that in Aitik six or seven double bench height (180 m in the hanging wall or 210 m in the footwall) is a stable interramp height, depending on hydrogeological conditions and rock type. As Salmijärvi and Liikavaara Östra are currently not planned to be deeper than 300 m, this interramp height is assumed to be valid at both locations (Sjöberg, et al. 2016). Thus the differences between the designs are in the particular bench face angles, interramp angles, and overall slope angles. In Figure 21 the sketch of the bench design parameters is presented.

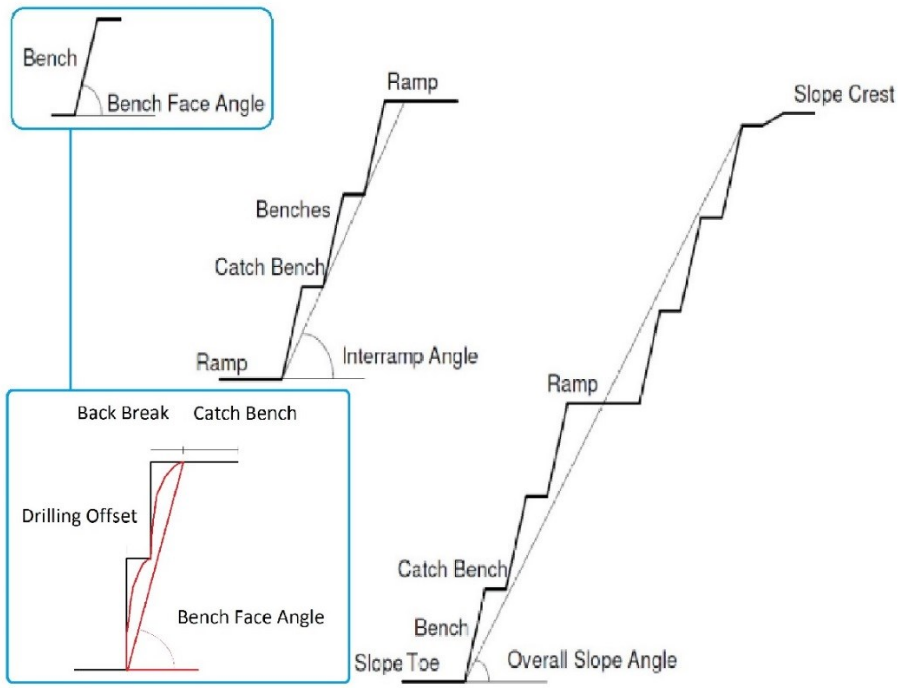

Figure 21 Bench design. After (Sjöberg, et al. 2016; Marklund, et al. 2007).

In Figure 22 the current slope design criteria for all sectors in the Aitik main pit are presented, where BFA is Bench Face Angle, IRA stands for Interramp Slope Angle, and OSA denotes the Overall Slope Angle (Sjöberg, et al. 2016).

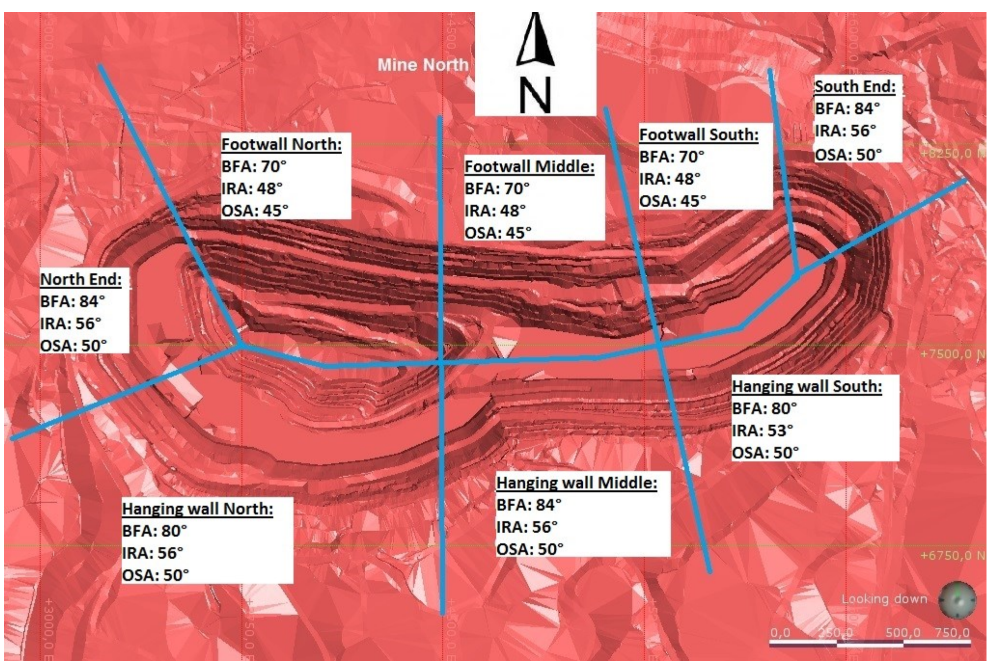

Figure 22 Design sectors with bench face, interramp and overall slope angles in Aitik after (Sjöberg, et al. 2016).

In Figure 23, the design sectors and the corresponding bench and interramp angles of Salmijärvi are shown. In all domains the overall slope angle (OSA) is 50° (Perks, 2015).

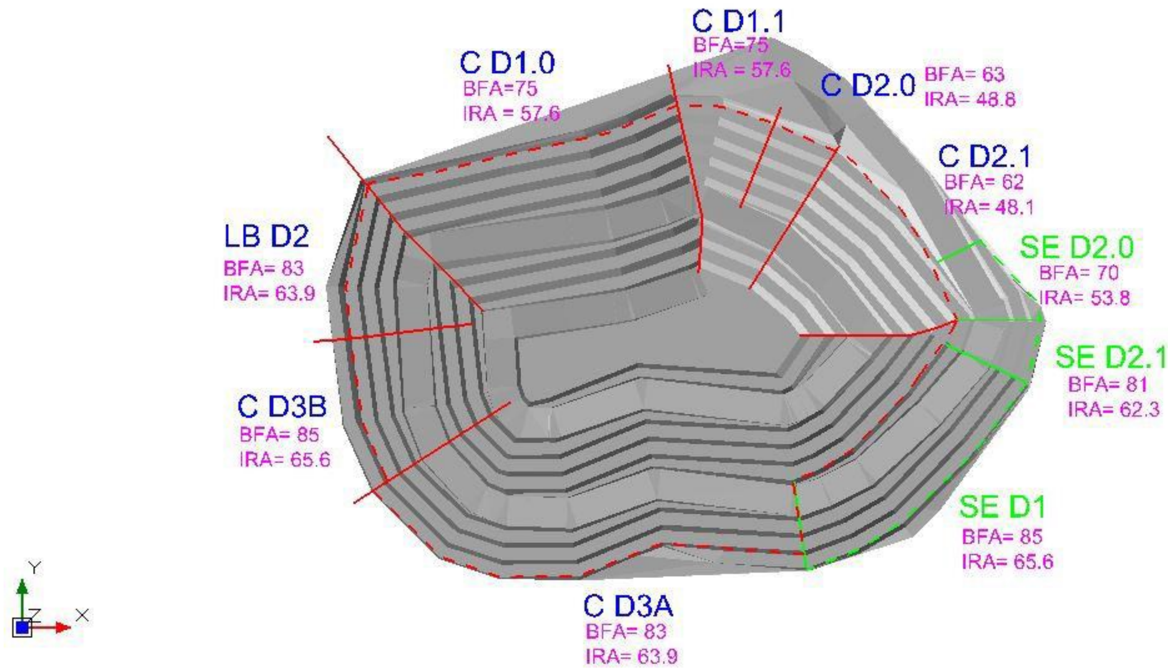

Figure 23 Bench face and interramp angles by design sectors in Salmijärvi (Perks, 2015).

In the conceptual study of Liikavaara Östra, several rock mechanical designs were conducted according to different mining scenarios. From the different setups the dimensions of the ´*Alternativ 1*` is presented here.

For the planned pit the following bench heights and angles have been calculated by Bergman in (2008a):

- Footwall OSA: 45°
- Hanging wall OSA: 50°
- Footwall IRA: 48°
- Hanging wall IRA: 56°
- Footwall and Hanging wall BFA: 80°

In the NE (North East) sector of the pit due to the presence of the crushed zones, different design criteria had been calculated. Flatter slope angles and lower bench heights are advised in order to maintain safe mine operation. Preferably 15 m high single benches, or 10 +10 m high double benches are recommended with 70° bench face angles. With these dimensions the interramp angles are 41° or 43° and the overall slope angles are 38° or 40° respectively (Bergman, 2008a). The described design elements are displayed in Figure 24.

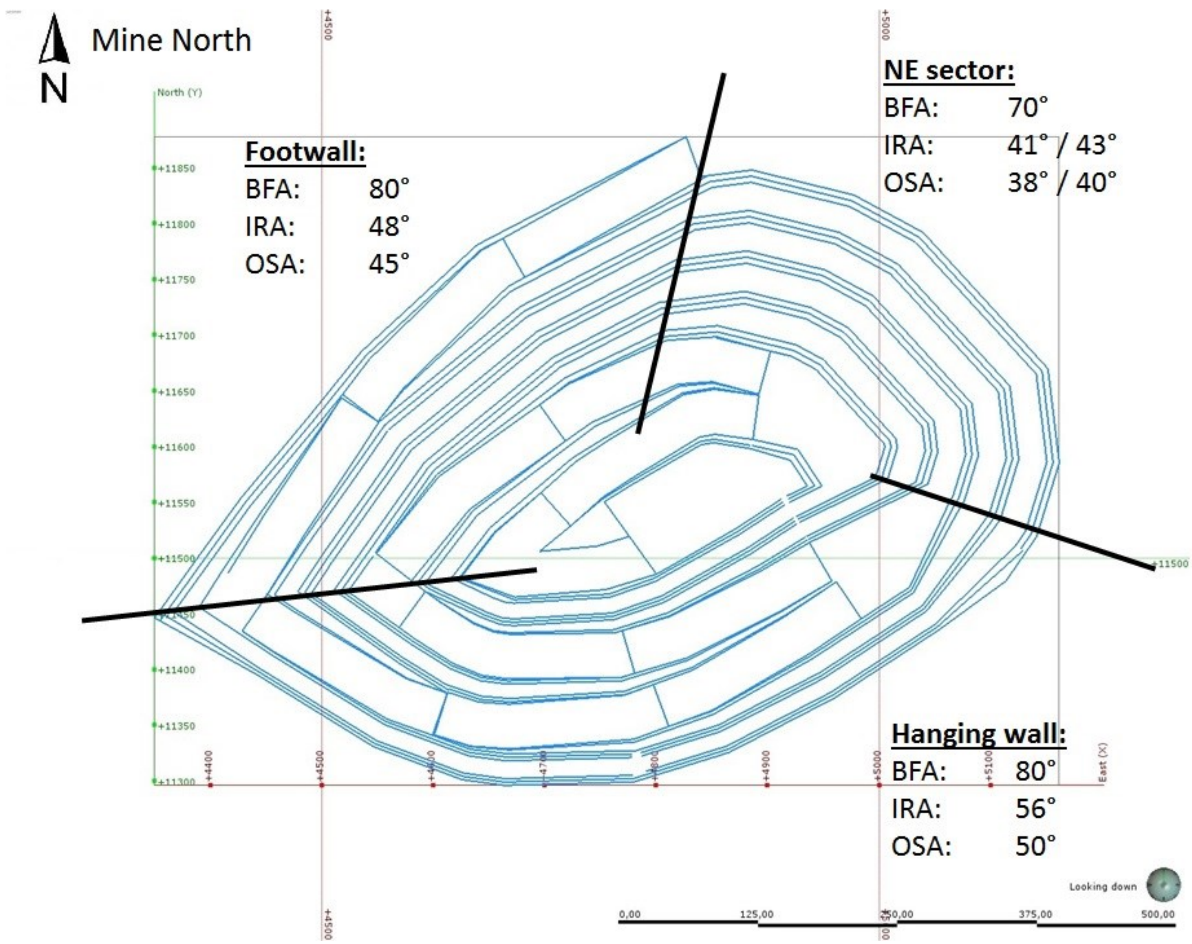

Figure 24 Design sectors in Liikavaara Östra after (Bergman, 2008a).

4. Data compilation and analysis

From the Liikavaara Östra area, extensive rock mechanical data is available from drill cores which are adequately covering the mineralization and its surroundings. In 2016, as part of the prefeasibility study of the project, a geological and rock mechanical drilling program commenced. During this program, the author of this thesis contributed to the work with logging of 942 m of drill core and conducting point load tests on 358 rock samples. In this chapter, the results of the 2016 drilling program and the previously available rock mechanical data are compiled and compared to the corresponding information from Aitik and Salmijärvi pits.

4.1. BRQD

BRQD data is available from all rock types in Liikavaara Östra with a total length of 10207.35 m of core. For the sake of clarity only those rock types are presented in this paper from which the percentage of drilled core is around or above 1% of the total sum of drilling (≥ 100 m). Figure 25 displays the results of the BRQD logging grouped by rock type; Table 8 and Table 9 present the comparison of the datasets to the previous Liikavaara Östra studies and the latest findings in Aitik and Salmijärvi. Note that in Aitik and Salmijärvi footwall and hanging wall rock types are differing from Liikavaara Östra, for this reason, the BRQD values of relevant footwall / hanging wall rock units are in the tables, thus comparison of the slope conditions is possible.

Table 8 shows that the recent findings (2016) and the ´historical` data (pre 2016) from Liikavaara Östra are matching despite the differences in the division of rock types, indicating that the results of the recent study are reliable. When the Liikavaara Östra BRQD values are compared to the rock mass quality of Aitik and Salmijärvi (Table 9) it is clear that the footwall side of Liikavaara Östra is in significantly worse condition than the other two pits, while the ore zone and the hanging wall side of Liikavaara Östra more or less are in the same condition based on the BRQD measurements.

Table 8 Liikavaara Östra BRQD results comparison.

Rock type	**BRQD** (Bergman, 2008a) **(Avg. + standard dev.)**	**RQD** (Krauland & Romedahl, 1996)	**Rock types**	**BRQD (Avg. + standard dev.)**
Biotite gneiss	N/A,	45 ± 28,	Biotite schist	46 ± 22
Biotite schist	47 ± 36	52 ± 22		
Volcanic sediment	37 ± 37	56 ± 24	Conglomerate	51 ± 24
			Turbidite	31 ± 27
Andesite, Epidote zone	52 ± 35	67 ± 16	Andesite	45 ± 23
			Granite	18 ± 25
			Diorite	59 ± 23
			Aplite	53 ± 18

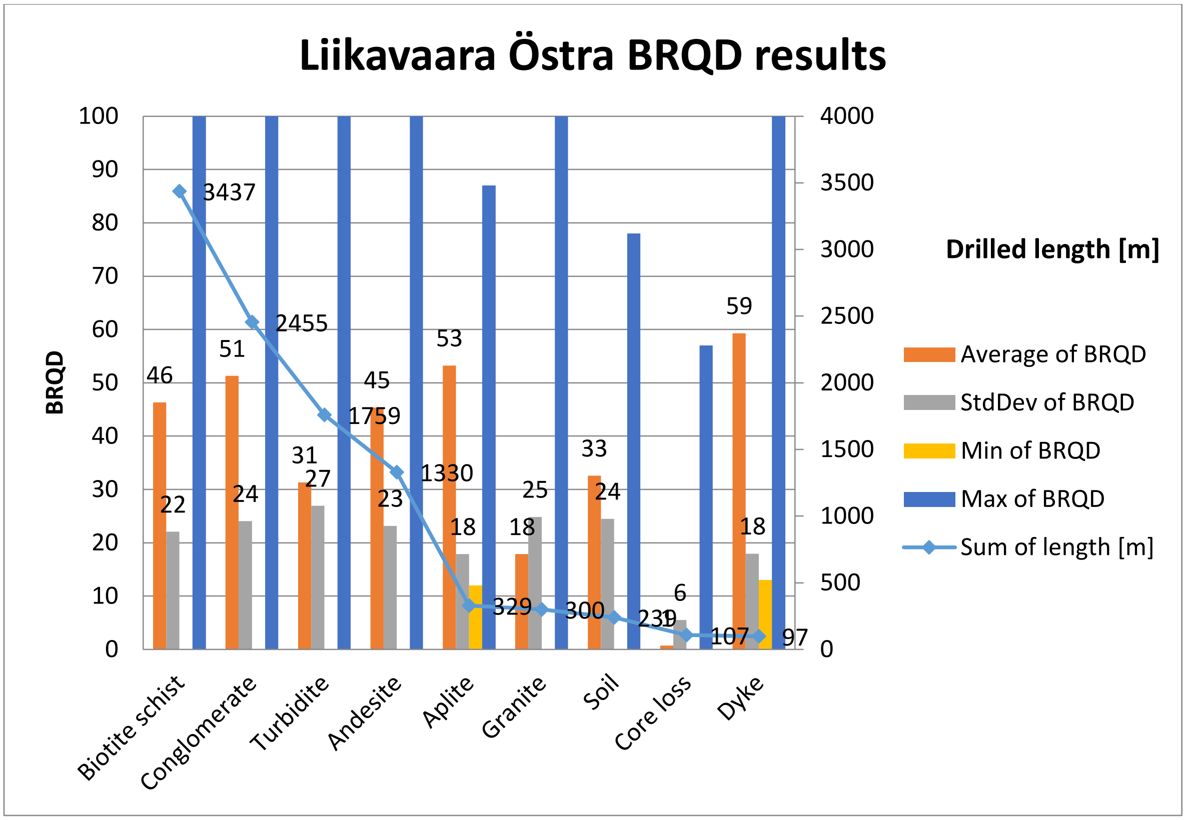

Figure 25 Liikavaara Östra compiled BRQD results.

Table 9 Liikavaara Östra, Aitik, and Salmijärvi BRQD comparison table. Green colored results indicate better rock conditions in Liikavaara Östra, while red values show worse circumstances than in Aitik or Salmijärvi.

		Liikavaara Östra	**Aitik** (Sjöberg, et al., 2016)	**Salmijärvi** (Bergman, 2008b)
Area	**Rock types**	**BRQD (Avg. + standard dev.)**	**BRQD (Weighted Avg.)**	**BRQD (Weighted Avg.)**
HW	Conglomerate	51 ± 24	55*	50*
Ore zone	Biotite schist	46 ± 22	52	-
	Biotite gneiss	-	62	69
	Andesite	45 ± 23	-	-
	Diorite	59 ± 23	58	39
	Aplite	53 ± 18	-	-
FW	Turbidite	31 ± 27	58**	39**
	Granite	18 ± 25		
	* Hornblende banded gneiss			** Diorite

4.2. RMR

In Liikavaara Östra total of 2967.8 m drill core have been logged according to the Boliden RMR standard, which is based on the 1976 version of the Rock Mass Rating (RMR_{76}). Figure 26 shows the results by units, Table 10 and Table 11 compare the findings to previous datasets of Aitik and Salmijärvi RMR information. It is clear from the presented data that rock mass (and joint surface and filling) conditions are slightly to significantly worse compared to Aitik and Salmijärvi.

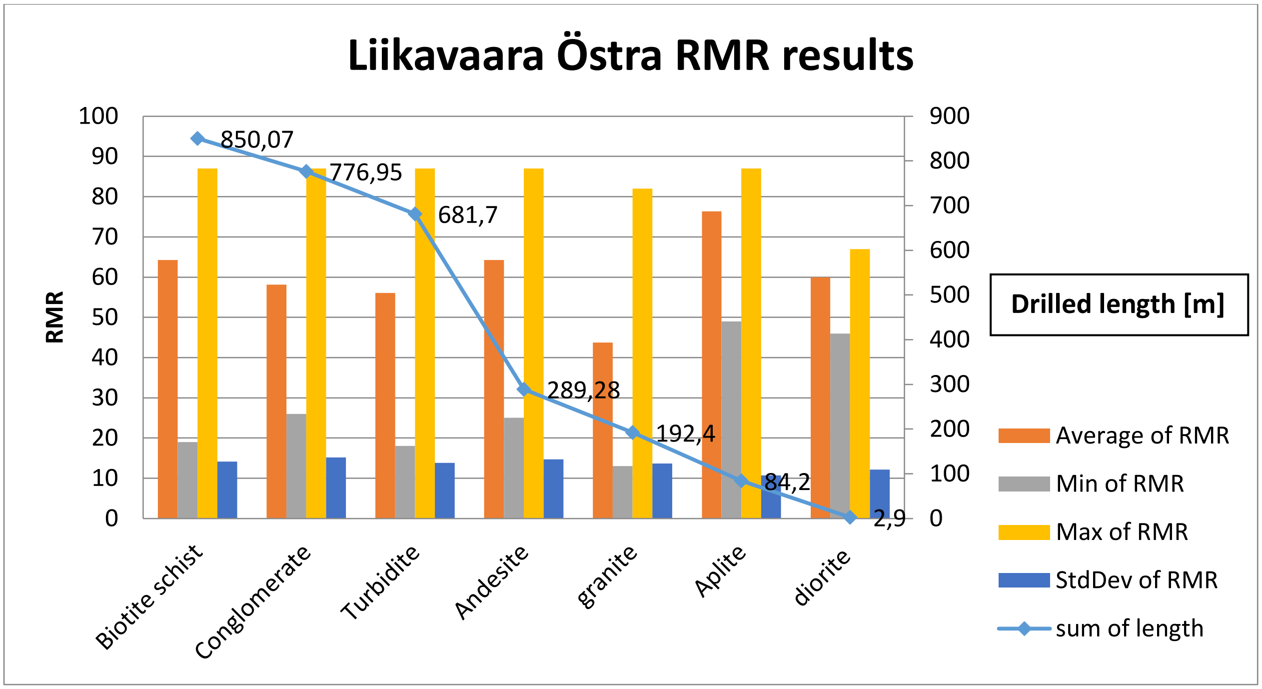

Figure 26 Liikavaara Östra RMR statistics

Table 10 Liikavaara Östra RMR statistics

<table>
<tr><th>Rock type</th><th>RMR (Bergman, 2008a)
(Avg. + standard dev.)</th><th>Rock types</th><th>RMR
(Avg. + standard dev.)</th></tr>
<tr><td>Biotite gneiss</td><td>N/A,</td><td rowspan="2">Biotite schist</td><td rowspan="2">64 ± 14</td></tr>
<tr><td>Biotite schist</td><td>70 ± 12</td></tr>
<tr><td rowspan="2">Volcanic sediment</td><td rowspan="2">67 ± 12</td><td>Conglomerate</td><td>58 ± 15</td></tr>
<tr><td>Turbidite</td><td>56 ± 14</td></tr>
<tr><td>Andesite,
Epidote zone</td><td>72 ± 12</td><td>Andesite</td><td>64 ± 15</td></tr>
<tr><td colspan="2" rowspan="3"></td><td>Granite</td><td>44 ± 14</td></tr>
<tr><td>Diorite</td><td>60 ± 12</td></tr>
<tr><td>Aplite</td><td>76 ± 11</td></tr>
</table>

Table 11 RMR comparison table of Liikavaara Östra, Aitik and Salmijärvi.
Red colored results indicate worse rock conditions in Liikavaara Östra, than in Aitik or Salmijärvi.

		Liikavaara Östra	Aitik (Sjöberg, et al., 2016)	Salmijärvi (Bergman, 2008b)
Area	**Rock types**	**RMR (Avg. + standard dev.)**	**RMR (Weighted Avg.)**	**RMR (Weighted Avg.)**
HW	Conglomerate	58 ± 15	67*	66*
Ore zone	Biotite schist	64 ± 14	64	-
	Biotite gneiss	-	70	76
	Andesite	64 ± 15	-	-
	Diorite	60 ± 12	69	60
	Aplite	76 ± 11	-	-
FW	Turbidite	56 ± 14	69**	60**
	Granite	44 ± 14		
* Hornblende banded gneiss ** Diorite				

BRQD and RMR values were also displayed in 3D modeling program, for which the *LEAPFROG GEO* (ARANZ Geo, 2016) software was utilized, see Figure 27. The presentation of rock mass properties this way enabled to identify the weak areas and discontinuity zones discussed earlier and to aid the process of selecting the design domain borders.

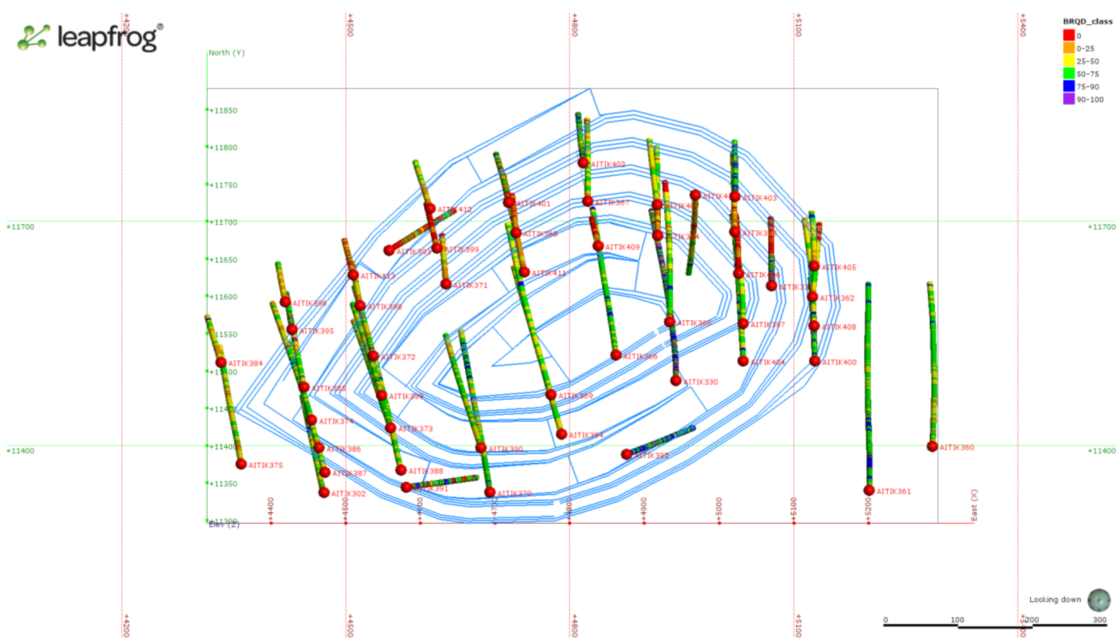

Figure 27 3D BRQD database with the final pit in Liikavaara Östra

4.3. BRQD-RMR correlation

As from the drill holes, only 29% of the total length was RMR logged, a BRQD-RMR correlation was calculated and displayed in the 3D database in order to enhance and aid the domain selection process. The correlation and extrapolation of BRQD values were based on a previous prefeasibility study for the Nautanen deposit (Itasca, 2016). In this work, RMR values were extrapolated to areas where only BRQD data was available. According to the insight of Itasca this correlation (with caution) can be used in the preliminary design stage, but extensive and careful data analysis must take place before application since rock mass and joint condition trends vary from site to site. The proposed correlation function of Itasca is displayed with a red dotted line in Figure 28.

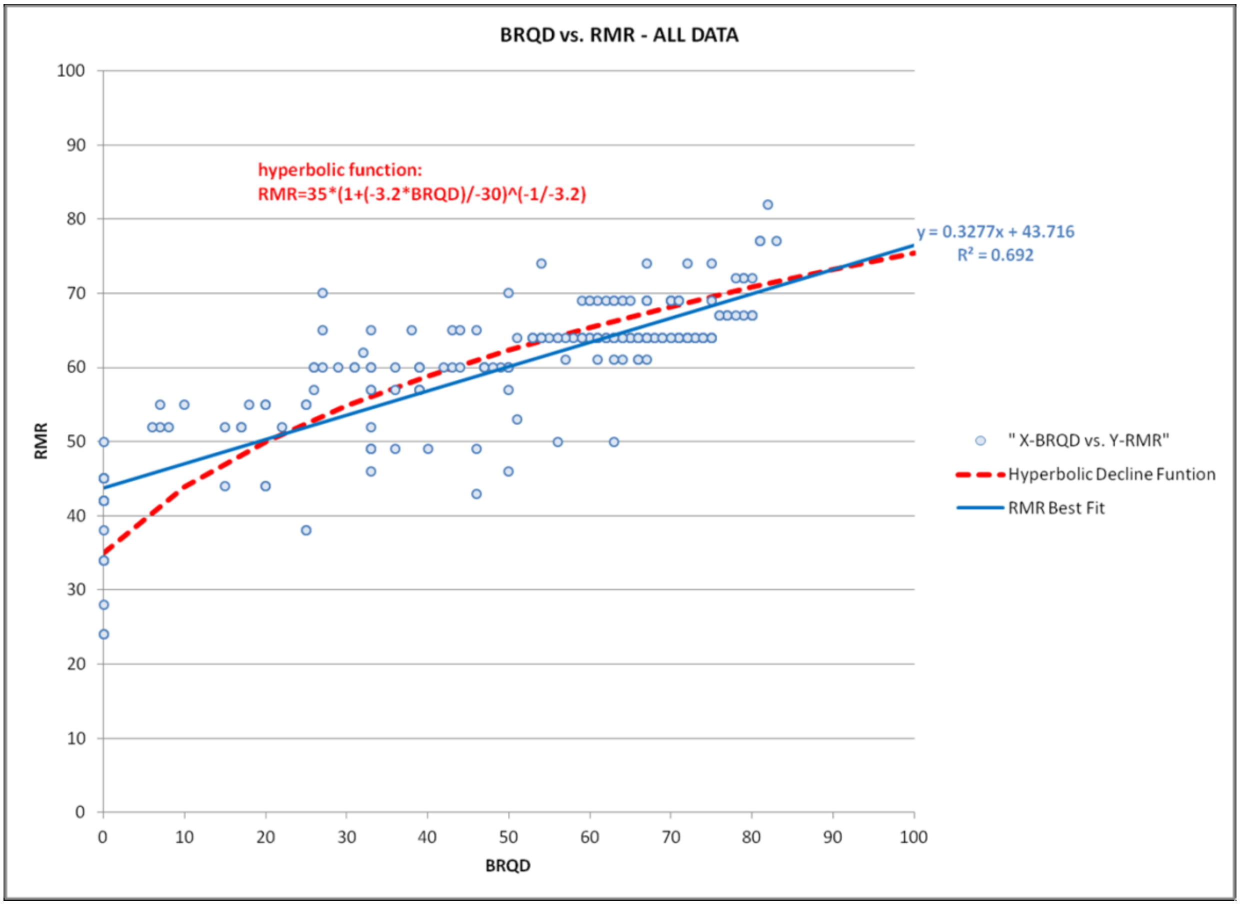

Figure 28 BRQD vs. RMR plot from the Itasca report (Itasca, 2016).

In the case of Liikavaara Östra, the RMR and BRQD were first plotted from drill holes where both data was available, after which the appropriate trend line was selected, which was fitting the plot the best possible way. The original trend line of Itasca (blue in Figure 29) would significantly overestimate the RMR values in Liikavaara Östra; this is why the function was modified (yellow, Figure 29) to cover the plot better. As the scatter of the RMR-BRQD plot is significant (approximately 20 RMR points), and the modified trend line runs in the average of it, the achieved results must be treated with caution and only used in the preliminary design stage. The extrapolated RMR results were only part of the domain selection process in this thesis, as an aid to better visualize the rock mass properties in the problematic (low BRQD value) areas, see (Figure 30).

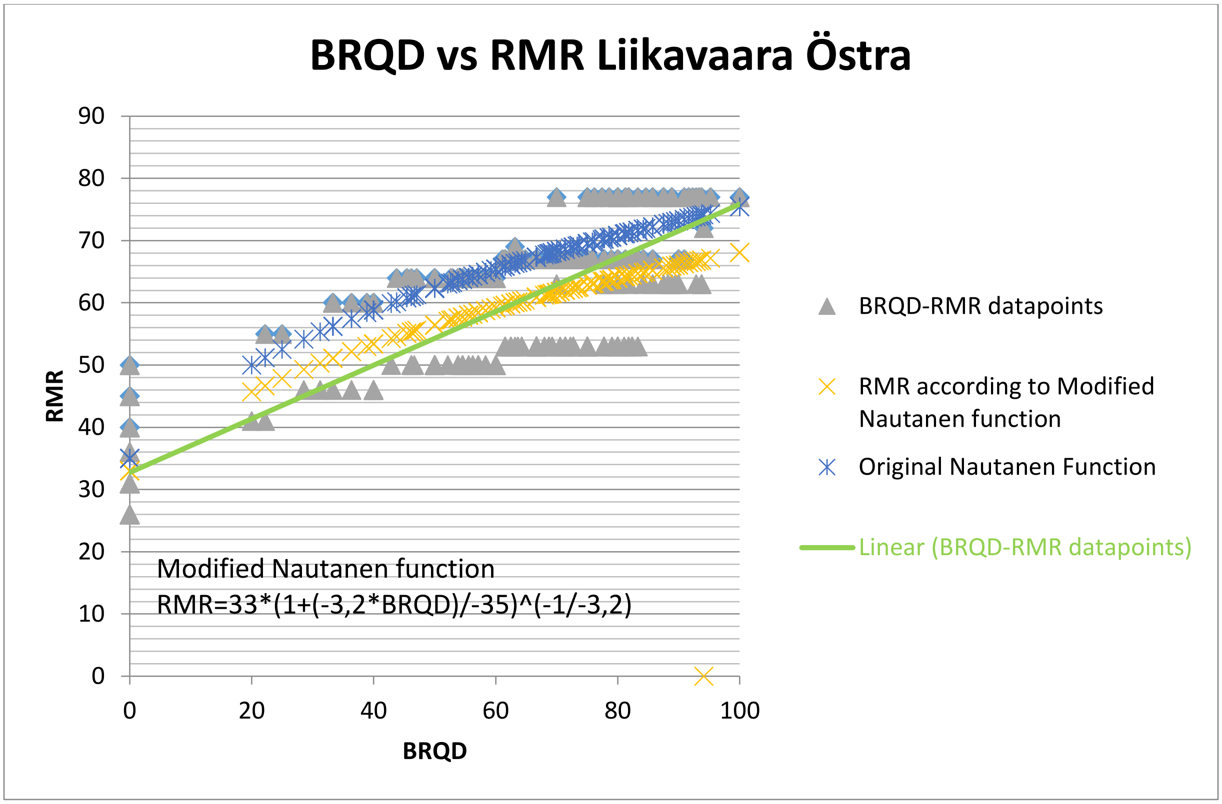

Figure 29 BRQD-RMR correlation for Liikavaara Östra.

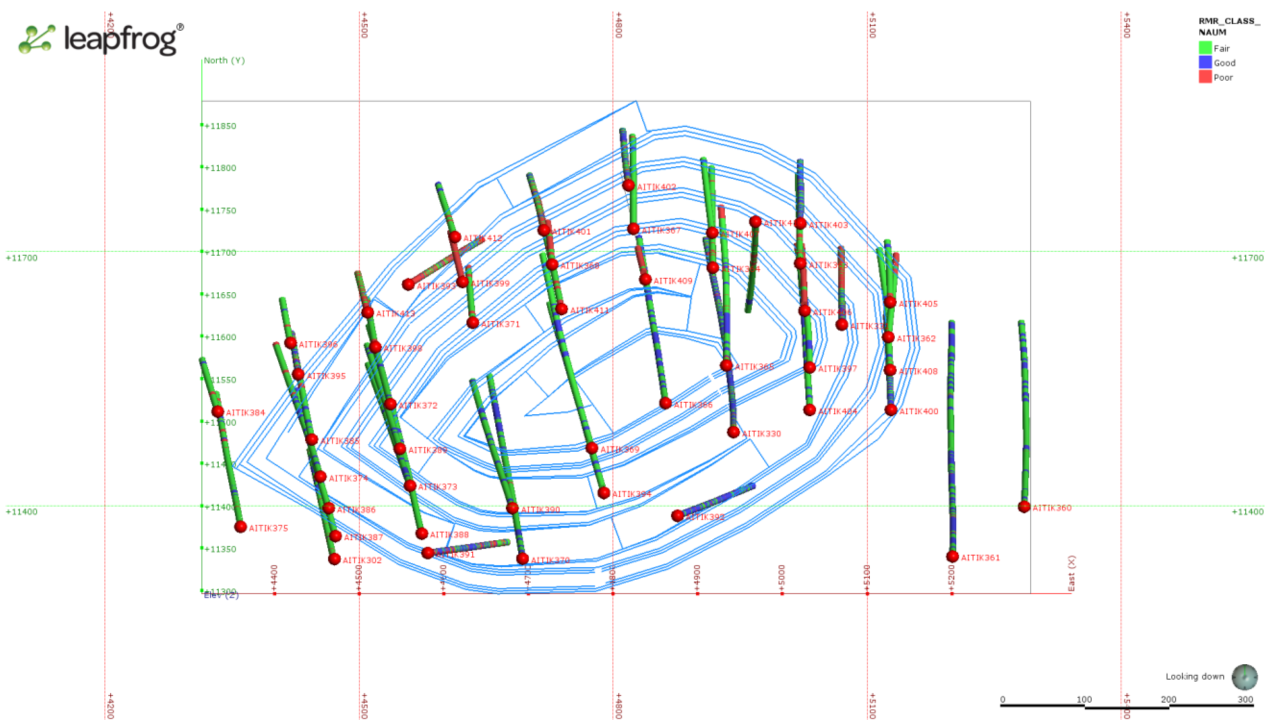

Figure 30 Extrapolated RMR values in Liikavaara Östra.

4.4. Compressive strength

For acquiring information regarding the compressive strength of rock types in Liikavaara Östra, point load tests (PLT) were executed. In the conceptual study of 1996, 306 samples were tested and in the present study, an additional 251 valid tests were conducted (out of 358 samples). During the measurements, the point load testing standards of the International Society of Rock Mechanics (ISRM) and Boliden Mineral AB were followed (ISRM, 1984; Boliden, 2000b). From the testing of the samples, first the point load index (I_S) was obtained, after size correction ($I_{s(50)}$) and conversion the uniaxial compressive strength (UCS) was received. The conversion factor between $I_{s(50)}$ and UCS was 22. Although previous reports show that the conversion factor of 22 is realistic in Aitik, UCS sampling and calibration of the factor for the rock types in Liikavaara Östra is recommended, thus more precise results can be obtained in further studies. Based on the orientation of the foliation in the core sample "a" (axial) or "d" (diametric) values of UCS results were recorded, for the definition of "a" and "d" see Figure 31. The results of the 2016 sampling campaign are shown in Figure 32 and Table 12. The comparison of the previous and recent point load tests of Liikavaara area is presented in Table 13.

Significant differences between the "a" and "d" values indicate that anisotropy (schistosity) is present in the sampled rock type (Figure 32, Table 12). In Liikavaara Östra this can be observed in all tested rock types, except the conglomerate, which is the main rock unit of the hanging wall and not considered to be foliated. Apart from the schistosity of the biotite schist (which is expected), these large differences of UCS values in the footwall rocks (granite and turbidite) also support the poor BRQD and RMR results indicated in the footwall side from logged core samples.

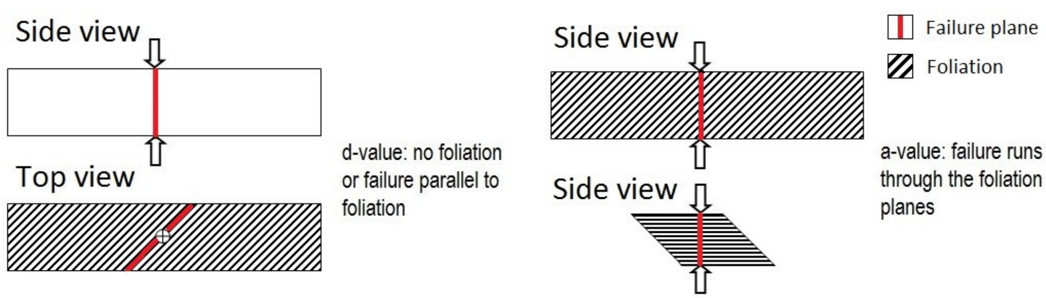

Figure 31 Definition of "a" and "d" values of failure according to the Boliden PLT standard (Boliden, 2000b).

Table 12 PLT test results in Liikavaara Östra.

PLT results by all rock types, "a" and "d" values					
Rock type	**Andesite**	**Biotite schist**	**Conglomerate**	**Granite**	**Turbidite**
Average "a" UCS [MPa]	**120.9**	**113.6**	**123.8**	**115.9**	**149.6**
Minimum "a" UCS [MPa]	29.2	34.6	23.1	59.0	72.7
Maximum "a" UCS [MPa]	170.0	240.2	243.2	152.4	280.7
Std. Deviation "a" [MPa]	53.0	55.7	55.5	37.4	47.8
Coefficient of Variation "a"	44%	49%	45%	32%	32%
Average "d" UCS [MPa]	**82.0**	**46.1**	**100.1**	**58.1**	**86.1**
Minimum "d" UCS [MPa]	47.0	6.1	15.6	3.8	6.1
Maximum "d" UCS [MPa]	130.7	97.7	187.5	170.5	173.8
Std. Deviation "d" [MPa]	29.9	26.1	48.9	70.4	45.6
Coefficient of Variation "d"	36%	57%	49%	121%	53%

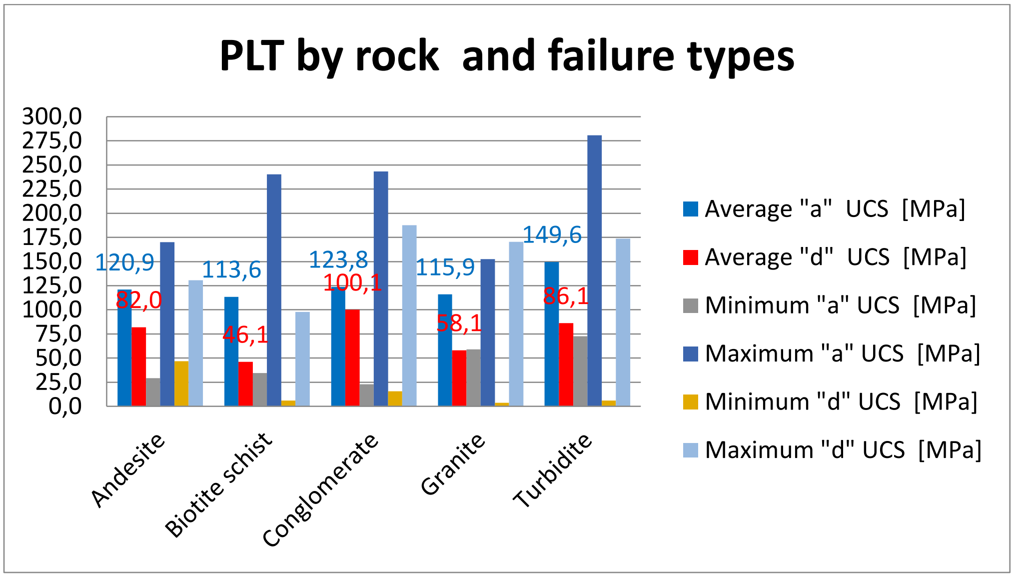

Figure 32 Point load test results in Liikavaara Östra.

Table 13 Comparison of point load test results in Liikavaara Östra.

Rock types	**PLT** (Krauland & Romedahl, 1996) **([MPa], Average ± Std. Dev.)**	**Rock types**	**PLT ([MPa], Average ± Std. Dev.)**
Biotite gneiss	77 ± 52	Biotite schist	62 ± 45
Biotite schist			
Volcanic sediment	150 ± 58	Conglomerate	108 ± 52
		Turbidite	105 ± 54
Andesite	157 ± 47	Andesite	96 ± 43
Epidote zone	145 ± 88		
		Granite	84 ± 62

From the comparison of the 1996 and 2016 test results, significant differences in UCS can be observed within the same rock types, with all rock units tested in 2016 having lower strength values than in the 1996 test. This can be the result of the accuracy of the point load test method, the difference in the testing device, the sampled rock or the different rock type nomenclature. Due to these differences, in this report, only the results of the 2016 sampling are used, which results in a conservative approach in the overall slope stability analysis of the Liikavaara Östra pit.

4.5. Joint orientation data

Joint orientation data are available from 7 video-filmed boreholes (marked black in Figure 33) and one partially core oriented drill hole (marked red in Figure 33) in Liikavaara Östra. It can be stated that joint orientation data is sufficiently probed in and around the pit, except the footwall side of the mine where borehole filming was not possible due to poor rock conditions. For this reason, one oriented core was targeted in the footwall area in 2016, but the orientation of core was not possible in full length due to excessively low rock quality. From the AITIK413 hole only 14.2 meter of core was possible to be oriented between borehole lengths of 49.3 m and 63.5 m, thus joint orientation data is limited and not comprehensive in this area. The stereonets of all holes in Liikavaara are in Appendix 3.

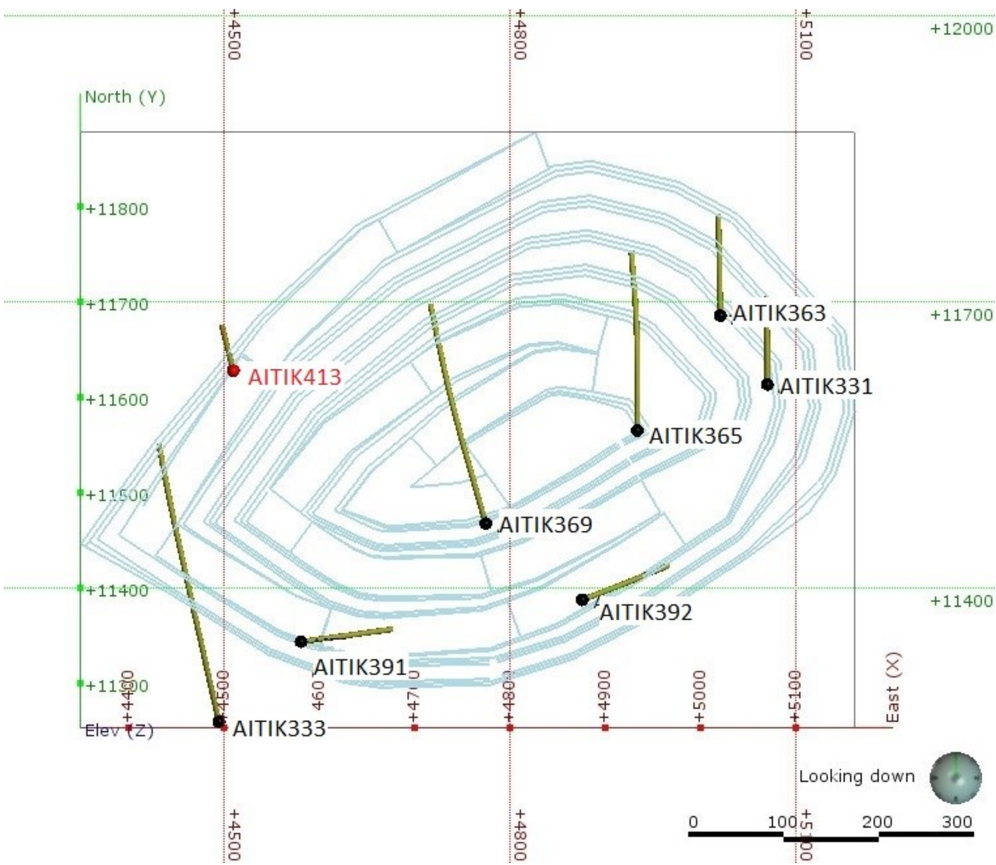

Figure 33 Location of joint orientation data sources in Liikavaara Östra. Black marked holes indicate borehole video mapping; red color shows hole with oriented core.

The joint orientation data was compiled into one database, hence the comparison with the 2008 and 2010 borehole video surveys is possible. In Figure 34 joint sets from 2008 (green), 2010 (blue) and 2016 (red) are projected; moreover, the strike of the mineralization is displayed with the thick red line. In Figure 35 the trend lines of the joint sets are presented over the geology of Liikavaara Östra. All sets determined are within a ±10° range when compared to the previous joint sets both regarding dip and dip direction. Only the dip directions of the horizontal sets are differing significantly, but in case of the horizontal set, it can be disregarded (Table 14, Figure 34).

Based on the 2016 borehole video results a new, fourth set was identified. The reason why this set could have been determined is the orientation of the rock mechanical holes drilled in 2016. While all previous holes were drilled perpendicular to the foliation of the ore zone, AITIK391 and AITIK392 were drilled roughly parallel to the foliation of the mineralization. This difference in drill hole orientation made this set became visible as the holes were not parallel to the N-S trending joint set. This location and direction of joint set 4 is concurring with the general joint orientation in Scandinavian ore mines. On average the following three sets are dominant and present in Scandinavia:

- Vertical, sub-vertical joint set (foliation of the rock)
- Horizontal, sub-horizontal set due to the Scandinavian uplifting since the last glacial period
- Orthogonal joint set which is perpendicular to the previous two sets caused by tectonic events.

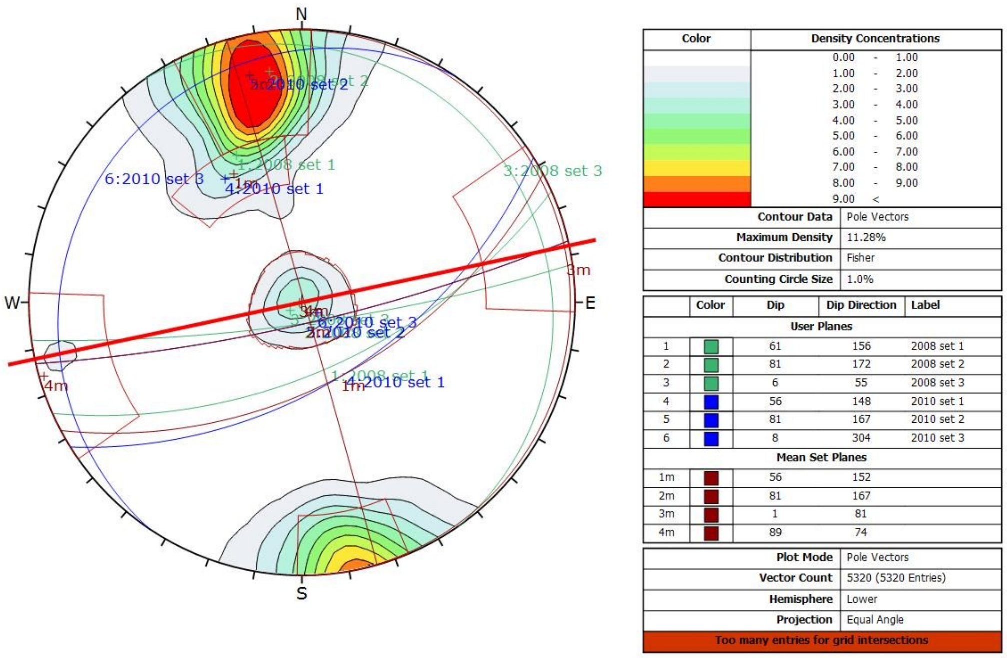

Figure 34 Comparison of 2008, 2010 and 2016 joint orientation data results.
The thick red line displays the trend of the ore mineralization.

It is vital to note here that this compilation of all data into one stereonet was not used in further analysis. In the separate design sectors, the relevant joint orientation data was used from the hole considered suitable for the domain.

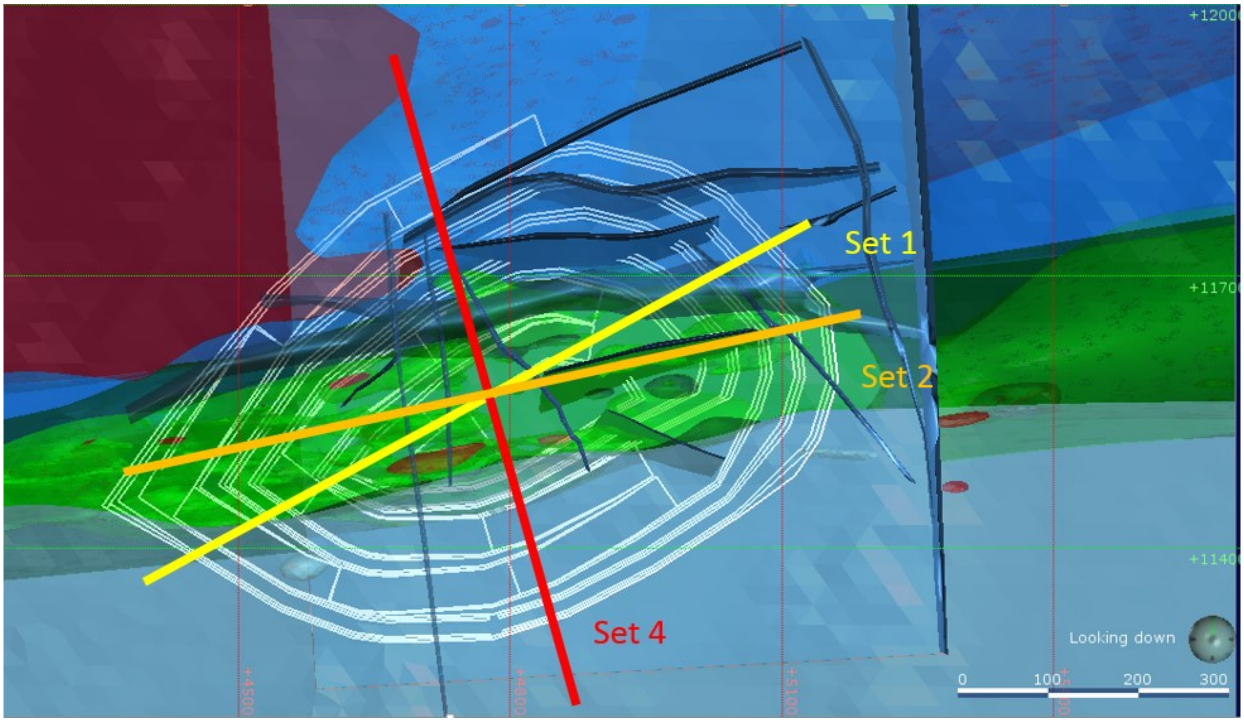

Figure 35 Joint set trend lines compared to the geology at Liikavaara Östra.

Table 14 Comparison of joint sets determined in 2008, 2010 and 2016.

Year	Set	Dip [°]	Dip direction [°]
2008	Set 1	61	156
2008	Set 2	81	172
2008	Set 3	6	55
2010	Set 1	56	148
2010	Set 2	81	167
2010	Set 3	8	304
2016	Set 1	56	152
2016	Set 2	81	167
2016	Set 3	1	81
2016	Set 4	89	74

The drill holes AITIK331 AITIK333, and AITIK369 (Figure 3-1, Figure 3-2, and Figure 3-5 in Appendix 3) are entirely or partially located in the biotite schist (ore zone). In all three stereonets two joint sets can be determined. Namely the steeply dipping foliation of the biotite schist and the (sub) horizontal set. However, in AITIK369 borehole a third set can be identified, which has an NE-SW trending rather similar to the orientation of the granite intrusion observed in the NW side of the pit (Figure 35).

AITIK363 and AITIK365 boreholes (Figure 3-3, and Figure 3-4 in Appendix 3) are located in an identified crushed area, giving fundamental information regarding the orientation of joints in these low rock quality zones. In both stereonets potential wedge forming joints can be determined, moreover in AITIK365 the steeply dipping foliation set and the (sub) horizontal discontinuity set are present, as the hole is partially located in the biotite schist. In AITIK363 the foliation and horizontal set caused by the uplifting are not identified.

AITIK391 and AITIK392 holes (Figure 3-6, and Figure 3-7 in Appendix 3) are the boreholes drilled in 2016 for borehole video mapping. In both holes several steeply dipping sets are present trending NNW-SSE (similar direction as the orthogonal set to the foliation and the (sub) horizontal set), W-E (parallel to the ore zone foliation) and NE-SW (trend of the granite intrusion). The horizontal set is also determined in both holes.

AITIK413 (Figure 3-8 in Appendix 3) hole was partially core oriented, as the rock quality was so low in the hole, that only 14.2 meters of the core were possible to be oriented. From this segment, only 23 joints were logged, from which two sets could be determined. Both sets are broadly concurring with the direction of the granite intrusion. It is possible that more joint sets are present in this area, but due to poor rock quality successful sampling was not possible until this date. It is strongly advised to obtain data from this segment of the planned pit, which is further eloborated upon in the Discussion chapter.

5. Domain determination for Liikavaara Östra

As rock type, intact rock strength, discontinuity orientation, and rock mass quality vary over the planned mine site, division of the pit into smaller sectors (domains) was necessary to execute effective and relevant slope stability analysis. The selection of the domains was based on rock type, intact rock strength, slope dip direction, discontinuity orientation, the presence of larger scale structures and previous design work.

In the latest available rock mechanic study of Liikavaara Östra, the pit was divided into three domains, namely Hanging wall, Footwall and North East see Figure 24 in Chapter 3. The further designation of domains was based on these sectors, moreover, the updated geological and rock mechanical information collected since the last study.

Although there are several rock types in Liikavaara Östra, only four of them are presented to such extent that it can significantly affect the slope stability. In this thesis, only the granite, the turbidite, the biotite schist and the conglomerate rock units were considered in the stability analysis process. The parameters of rock types used are shown in Figure 36.

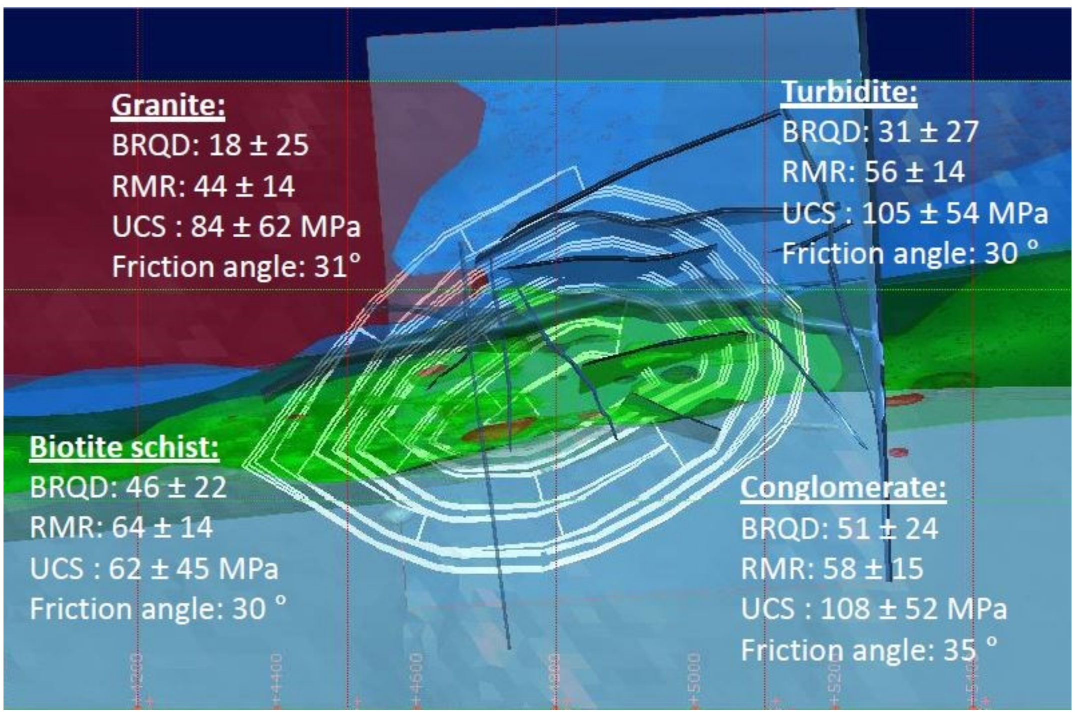

Figure 36 Rock units considered at the slope design process.
The friction angles denote the joint friction angle of the rock type.

By investigating the strength and quality values of the rock units (BRQD, RMR, and UCS), it was clear that the division of the sectors according to the geology was necessary. Rock types of the footwall (granite and turbidite) had lower BRQD and RMR values than the ore zone (biotite schist) and the conglomerate in the hanging wall. The differences in the UCS and friction angle values also required the separation. The foliation present in the biotite schist, granite and turbidite also support the division of domains.

The previously described (see subchapter 2.3) large scale structures (crushed zones) can also affect the slope stability. As these zones are almost exclusively in the North East and Footwall domains, it was reasonable to divide the pit into smaller sectors in these areas, to investigate the effects of these zones in higher resolution.

Based on the differing BRQD, RMR and UCS values of the rock types, moreover the joint and discontinuity zone orientation data, the pit can be divided into the design domains listed in Table 15 and visualized in Figure 37. The stereonets of the joint orientation data with the corresponding domains are displayed in Figure 38.

Table 15 Slope stability design domains of Liikavaara Östra.

Domain name	Domain code	Main rock type	Slope Dip direction [°]
Hanging wall 1	HW1	Conglomerate	319
Hanging wall 2	HW2	Conglomerate	9
West End 1	WE1	Biotite schist	30
West End 2	WE2	Biotite schist	130
Footwall 1	FW1	Granite	146
Footwall 2	FW2	Turbidite	156
North East 1	NE1	Turbidite	185
North East 2	NE2	Turbidite	217
East End	EE	Biotite schist	274

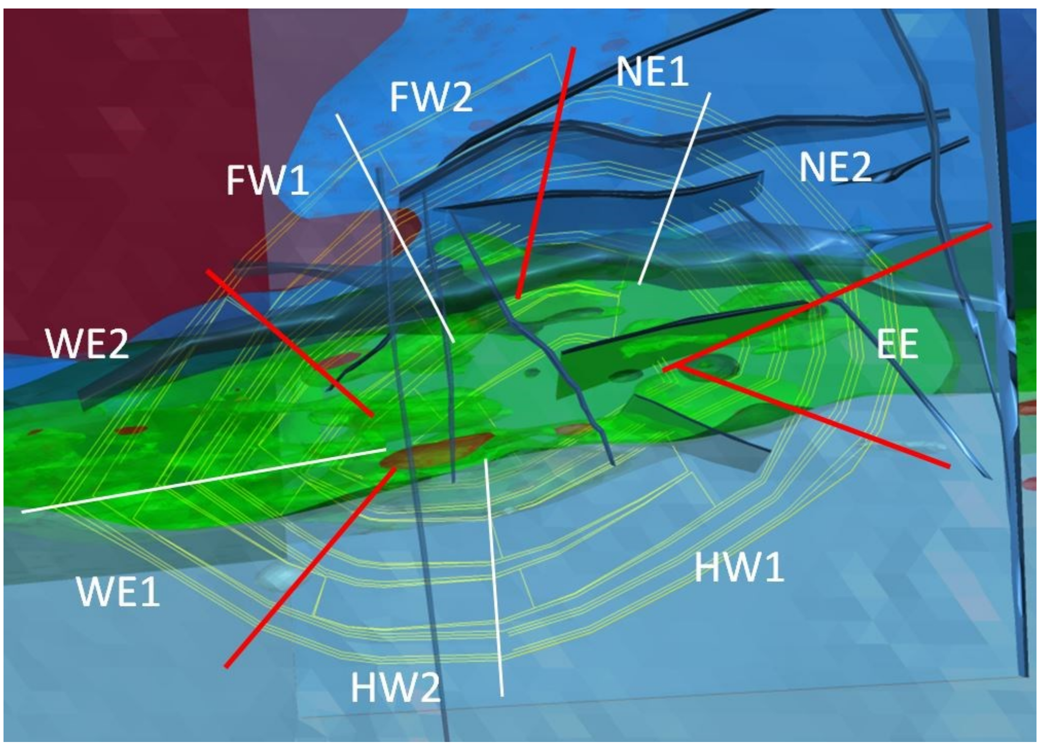

Figure 37 Final design domains of Liikavaara Östra with the pit outline, geology and identified discontinuity zones.

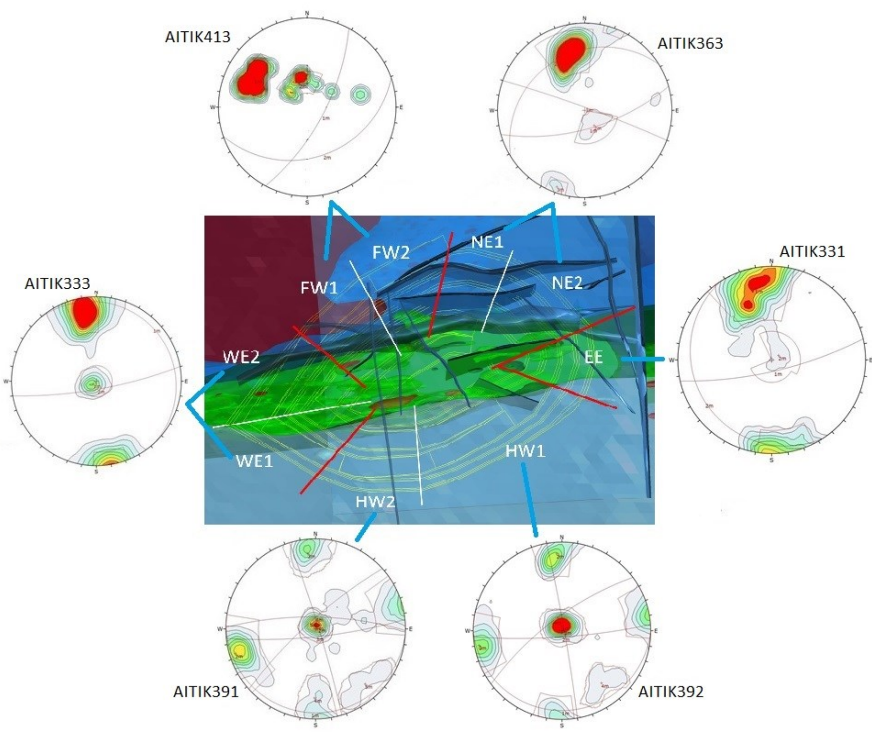

Figure 38 Final domains in Liikavaara Östra with relevant stereonets.

During the design process not only the abovementioned final domains were used. Temporary design sectors were also utilized in certain areas, in order to fully understand and assess bench stability. These temporary or working domains were situated in the Footwall side of the pit where the presence of crushed zones and other structures were evident and additional investigation was required. The results of the temporary sectors were implemented to the final design recommendations, and the domains were merged with the main sectors. The temporary domains are displayed in Figure 39.

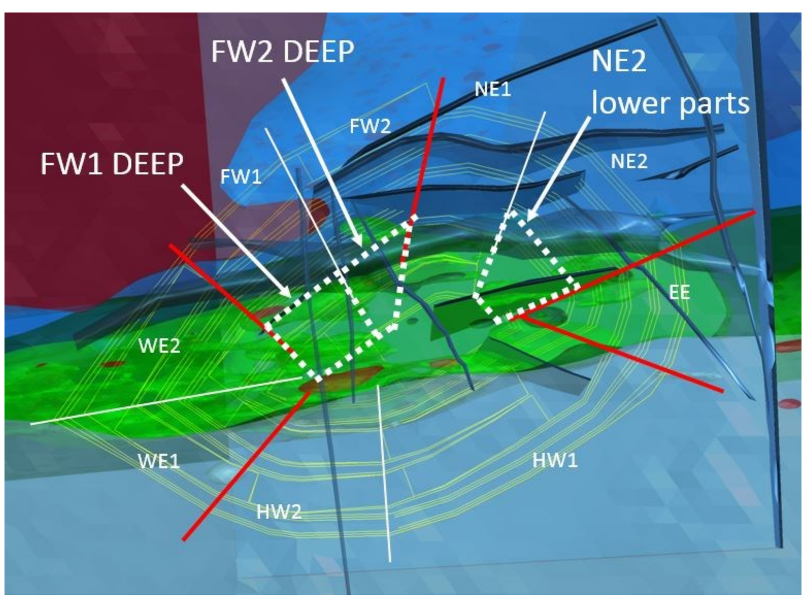

Figure 39 Location of temporary domains in Liikavaara Östra.

6. Bench slope stability analysis

In hard rock open pit mines, bench scale failures are governed by the presence, orientation and surface properties of discontinuities. Depending on the relative direction of the bench slope and the joint sets, either planar, wedge or toppling failures can develop on bench scale. In this chapter, first the mentioned failure modes are assessed in each domain with kinematic analysis. Afterwards the selected sectors are further analyzed with probabilistic and/or deterministic methods. Then based on the results of the various analyses, and experiences gathered in Aitik and Salmijärvi, bench (and interramp) design parameters for Liikavaara Östra mine are recommended.

6.1. Failure types

The major types of discontinuity driven failures in hard rock mines are the planar, wedge and toppling failures, where the main influencing factor is the orientation of joint sets compared to bench slope direction. In this subchapter, all three types are briefly assessed and in the later chapters are referenced.

6.1.1. Planar failure

For planar failure to occur in benches, several criteria must be satisfied (Wyllie & Mah, 2004), see Figure 40. These are the following:

- The sliding plane (existing joint surface or formed plane) must be parallel to the dip direction of the bench or must be within a ±20° window.
- The dip of the plane must be lower than the dip of the bench slope (the plane must daylight) $\psi_f > \psi_p$
- The dip of the plane must be larger than the friction angle calculated (or assumed) for the joint (sliding plane) surface $\psi_p > \phi$
- The sliding plane intersects the upper slope or ends in a tension crack

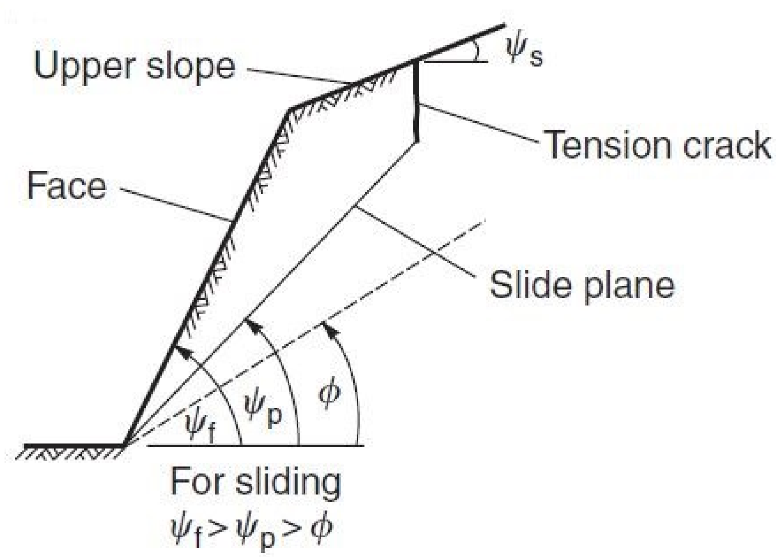

Figure 40 Criteria for planar failure, after (Wyllie & Mah, 2004).

6.1.2. Wedge failure

For wedge failure to occur the following circumstances must be achieved:

- Two failure planes must intersect in the bench, and the line of intersection must daylight in the slope, see Figure 41 a
- The dip of the intersection line must be larger than the friction angle of the two joints and flatter than the dip of the bench ψfi > ψi > ϕ, see Figure 41 b

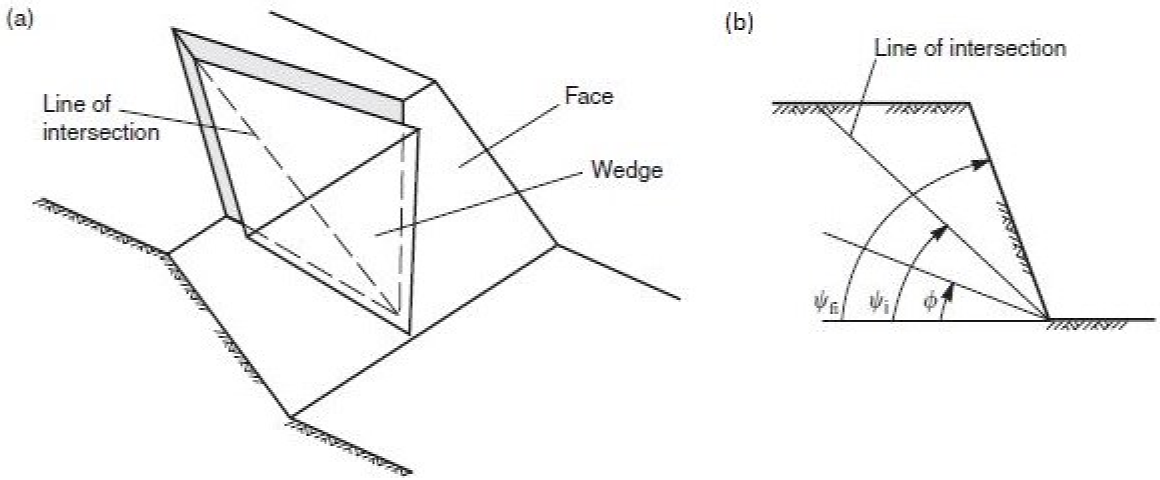

Figure 41 Criteria for wedge sliding, modified after (Wyllie & Mah, 2004).

6.1.3. Toppling failure

Within toppling failure two major types of failure can be identified, namely, block and flexural toppling. Block toppling is mostly typical to vertically, sub-vertically jointed rock mass with orthogonal sets, flexural toppling is more specific to steeply dipping, thin bedded formations where interlayer slip occurs. The combination of these two types is also possible.

Block toppling

Block toppling occurs in hard rock conditions when at least two defined sets of joints are present. The necessary discontinuities for block toppling to occur are the set (or sets) of discontinuities that are steeply dipping into the bench face forming rock columns; and a widely spaced, orthogonal joint set which cuts into the steep rock columns. Smaller columns at the bottom of the slope are pushed forward by the larger rock columns which are overturning, resulting in further toppling, see Figure 42.

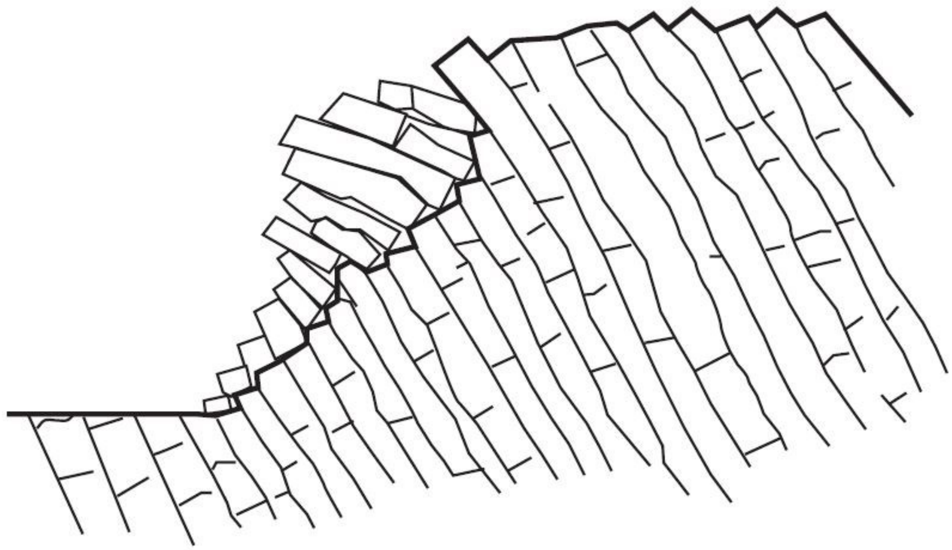

Figure 42 Sketch of direct (block) toppling. Steeply dipping rock columns are undercut by widely spaced orthogonal joints. Goodman and Bray (1976) as cited in (Wyllie & Mah, 2004).

Flexural toppling

Flexural toppling develops in rock slopes with steeply dipping discontinuities, where the orthogonal joint set is not well developed. In this case, the failure occurs as an interlayer slip, which develops between the rock layers. If the dip of the bedding planes is δ, ϕ is the joint friction angle, and the bench face angle is α, then interlayer slip occurs if (90- δ) + ϕ < α (Goodman, 1980), see Figure 43.

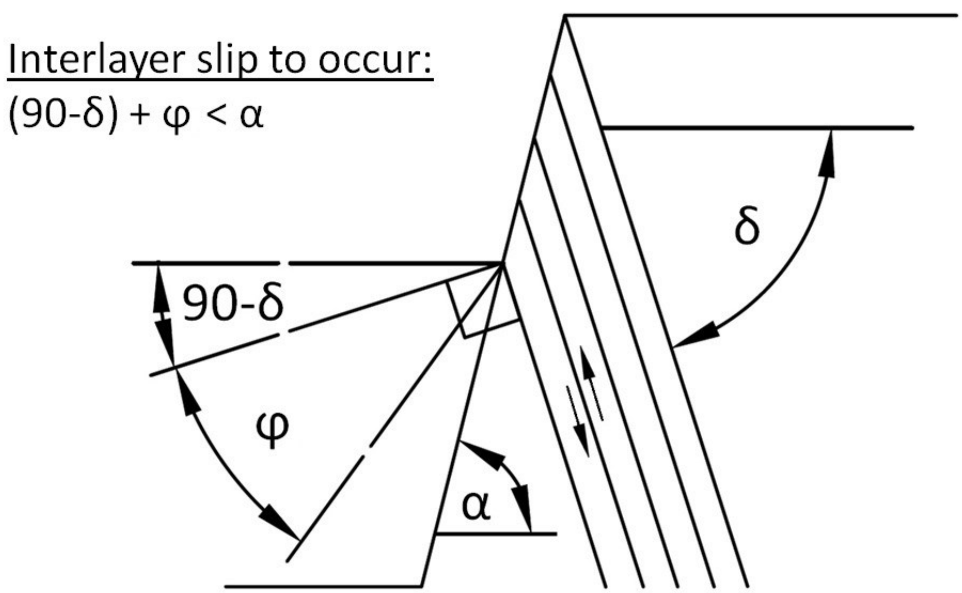

Figure 43 Sketch of flexural toppling criteria after Goodman (1980).

6.2. Bench width design criteria

In the design criteria of Liikavaara Östra (similarly like in Aitik and Salmijärvi) the width of the catchment bench was calculated based on the empirical equation developed by Ryan & Pryor in 2000, see Equation 1. This criteria relies on the Ritchie (1963) criteria as cited in (Read & Stacey, 2008).

Equation 1 Catch bench width criteria.

$$\boldsymbol{Bench\ width\ (m) = 0.2 * bench\ height\ (m) + 4.5\ m}$$

In the conducted analysis with 30 m and 15 m bench heights, 11 and 8 meters wide catchment bench widths were used respectively. In both analysis, this bench width was denoted as the "effective bench width", which is not the excavated bench width. As the mining will be executed by drilling and blasting, back break from blast damage is expected. Thus the blasted bench width is wider and includes the blast damaged zone (where the back break occurs) and the effective bench width. In the estimated back break a 3 m wide drilling offset was included for the hanging wall side, and a 5.5 m offset distance for the footwall side. This offset value is required by the regulations and limitations of the blast hole drilling process. The bench dimensions are visualized in Figure 44.

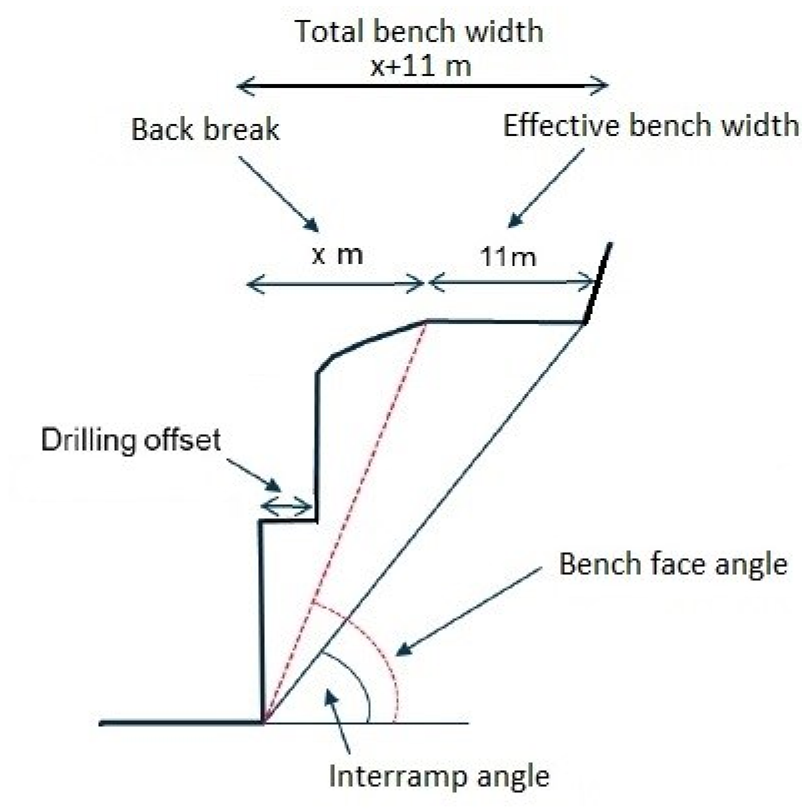

Figure 44 Bench dimensions in Liikavaara Östra modified after Perks (2015). Note that the drilling offset required by production criteria is included in the estimated back break.

6.3. Kinematic analysis

To identify the critical joint sets and failure types in each domain, first, the kinematic analysis of the joint orientation data was conducted with *DIPS 7.0* (Rocscience, 2016b) software. For each domain the corresponding historic or newly acquired datasets (stereoplots) were used alongside with the determined bench slope dip directions and friction angles. The bench face angles and interramp angles tested in the analysis were based on the results of the latest rock mechanical design criteria of Liikavaara Östra from 2008. The kinematic analysis was executed with the built-in kinematic analysis tool of *DIPS 7.0.* Figure 45 displays the kinematic analysis applied for planar failure case in the HW1 domain. In Table 16 the results of the analysis are presented for planar failure in the final domains. For the temporary sectors the results for planar failure are in Table 17. The results of the kinematic analysis for the rest of the failure types (wedge, direct toppling and flexural toppling) are presented in Appendix 4.

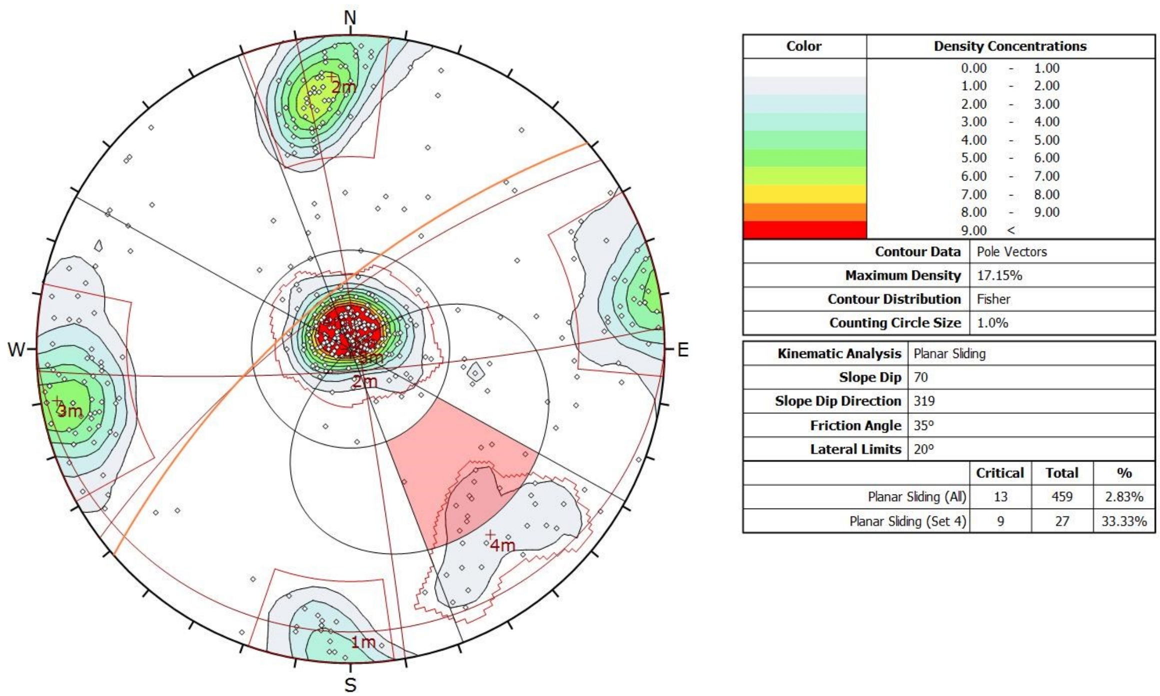

Figure 45 Stereoplot of kinematic analysis for HW1 domain, planar failure case.

Table 16 Kinematic analysis for final domains, planar failure case.

Kinematic analysis - PLANAR FAILURE						
Domain	Slope Dip Direction [°]	Type of Angle	Slope Angle [°]	Friction Angle [°]	Percentage of Total Poles [%]	BOREHOLE
WE1	30	BFA	70	30	0.7	AITIK333
			80	30	1.1	
			90	30	2.1	
		IRA	48	30	0.5	
WE2	130	BFA	70	30	2.1	AITIK333
			80	30	3.5	
			90	30	4.4	
		IRA	48	30	0.5	
EE	274	BFA	70	30	0.7	AITIK331
			80	30	0.7	
			90	30	1.4	
		IRA	48	30	0.7	
FW1	146	BFA	70	31	17.4	AITIK413
			80	31	30.4	
			90	31	30.4	
		IRA	48	31	13.0	
FW2	156	BFA	70	30	21.7	AITIK413
			80	30	21.7	
			90	30	21.7	
		IRA	48	30	17.4	
NE1	185	BFA	70	30	15.6	AITIK363
			80	30	30.9	
			90	30	35.6	
		IRA	38	30	0.9	
NE2	217	BFA	70	30	1.2	AITIK363
			80	30	1.7	
			90	30	3.0	
		IRA	38	30	0.1	
HW1	319	BFA	70	35	2.8	AITIK392
			80	35	5.0	
			90	35	7.0	
		IRA	56	35	0.4	
HW2	9	BFA	70	35	3.2	AITIK391
			80	35	5.1	
			90	35	8.0	
		IRA	56	35	1.3	

Table 17 Kinematic analysis for temporary domains, planar failure case in Liikavaara Östra.

Kinematic analysis - PLANAR FAILURE Temporary domains						
Domain	Slope Dip Direction [°]	Type of Angle	Slope Angle [°]	Friction Angle [°]	Percentage of Total Poles [%]	BOREHOLE
NE2 lower parts	217	BFA	70	30	1.2	AITIK365
			80	30	2.0	
			90	30	2.5	
		IRA	38	30	0.5	
FW1 - DEEP	146	BFA	70	31	13.5	AITIK369
			80	31	29.5	
			90	31	61.9	
		IRA	48	31	0.4	
FW2 - DEEP	156	BFA	70	30	10.7	AITIK369
			80	30	27.1	
			90	30	61.5	
		IRA	48	30	0.8	

Failure types were further analyzed with probabilistic and/or deterministic analysis if the percentage of critical poles were larger than 10% of the total poles. In particular cases (HW1 and HW2) where the percentage of total poles was close but below the 10% threshold limit; but the failure of one set was significant, further analysis was also conducted on the failure type in question. Based on the results of the kinematic analysis of the final and the temporary domains, the design sectors were further analyzed for the following failure types:

- Wedge failure: All domains (probabilistic and deterministic analysis)
- Planar failure: FW1, FW1 DEEP, FW2, FW2 DEEP, NE1, HW1 and HW2 (probabilistic and deterministic analysis)
- Toppling failure (direct toppling): EE and HW1 (deterministic analysis only)

6.4. Probabilistic analysis

6.4.1. Introduction to probabilistic analysis

The uncertainty of design parameters is inevitable in rock engineering as the subject of design is the heterogeneous rock mass, where material properties have uncertainty and variability. Moreover, it is not unique that the amount of data available is insufficient due to limited sampling methods, inaccessible location or merely financial considerations. To overcome these challenges for decades, the deterministic analysis methods were used to assess slope stability. These methods use the mean value of the available datasets or single estimated amounts as input values; giving the results in the term of Factor of Safety (FoS) which is the ratio between the resisting and driving forces. According to the theory, the investigated slope is stable if the FoS is at least 1.0. Based on experience in the mining industry and civil engineering larger than 1.0 FoS values are used in slope analysis, due to the uncertainties of data, consequences of failure and limitations of the used design theory. Over time authors recommended several FoS values based on the purpose of the slopes. In Figure 46 the industry-wide accepted ranges of FoS values are presented by Priest & Brown (1983) as cited in (Read & Stacey, 2008).

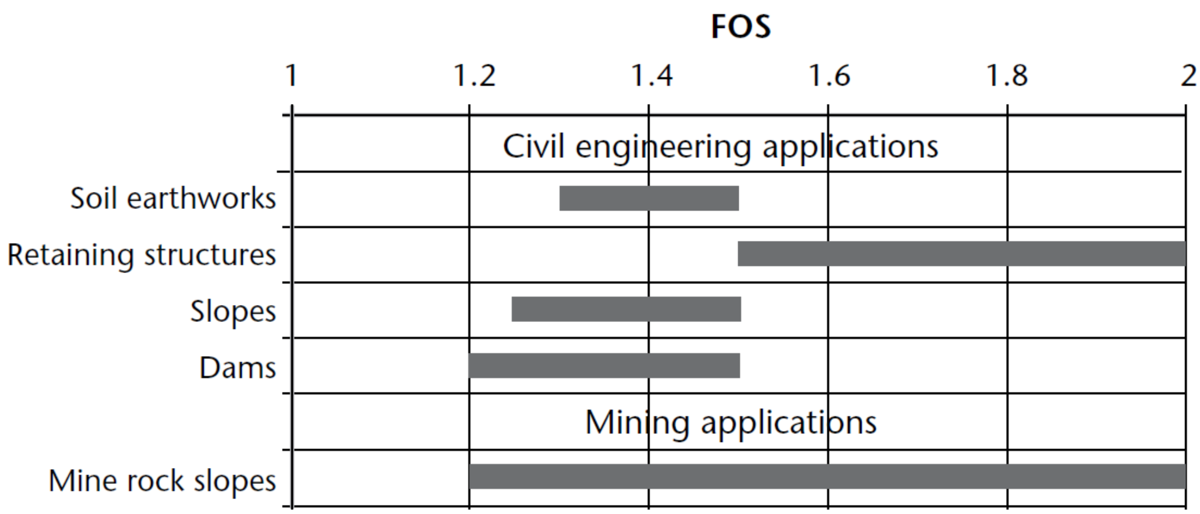

Figure 46 Examples of FoS values from Priest & Brown (1983) as cited in (Read & Stacey, 2008).

Because of these abovementioned reasons over the years, methods were introduced to quantify the uncertainty of some of the input data. These design processes use multiple realizations to calculate the Probability of Failure (PoF). There are two options to get the PoF of a particular slope:

1. The FoS is set as a random variable, and the probability of FoS being smaller or equal 1.0 is assessed.
2. The probability of the demand (driving forces) exceeds the capacity (resisting forces) is investigated.

From these two options, the first is used widely and was applied in this study as well. The distinct advantage of the probabilistic slope analysis is that not only one but multiple realizations are calculated giving the opportunity to take into account large number of possibilities (Read & Stacey, 2008).

For the design guidelines of PoF the following values (Table 18) are acceptable as cited in (Read & Stacey, 2008).

Table 18 FoS and PoF design values from Priest & Brown (1983) as cited in (Read & Stacey, 2008).

Consequence of failure	Examples	Acceptable values		
		Mean FoS	Minimum P[FoS < 1.0]	Maximum P[FoS < 1.5]
Not serious	Individual benches; small (< 50 m), temporary slopes, not adjacent to haulage roads	1.3	10%	20%
Moderately serious	Any slope of a permanent or semi-permanent nature	1.6	1%	10%
Very serious	Medium-sized (50–100 m) and high slopes (<150 m) carrying major haulage roads or underlying permanent mine installations	2.0	0.30%	5%

To execute the probabilistic analysis, different simulation methods are available and widely used in slope stability calculations such as the Monte Carlo simulation, first-order second-moment method and point estimate method. In the conducted analysis, the built-in probabilistic analysis tool of the design programs were used, and Monte Carlo simulation was applied. The estimation of the required number of simulations was calculated as follows. Based on Equation 2 (Gibson, 2011) the necessary amount of iterations can be computed, although two facts must be considered:

1. The number of MC simulations is not a function of the number of random input variables
2. PoF must be known before simulation

Equation 2 Equation for estimating the number of simulations.

$$n = \left(\frac{d}{\alpha}\right)^2 \frac{1-p}{p}$$

Where:

- n = Number of simulations
- d = Estimated normal standard deviate
- α = Acceptable error in analysis
- p = PoF

To estimate the normal standard deviate based on the confidence level, values from Table 19 were used.

Table 19 Normal standard deviate (Gibson, 2011).

Percentage of Confidence (%)	Normal Standard Deviate (d)
80	1.28
85	1.44
90	1.64
95	1.96
99	2.57

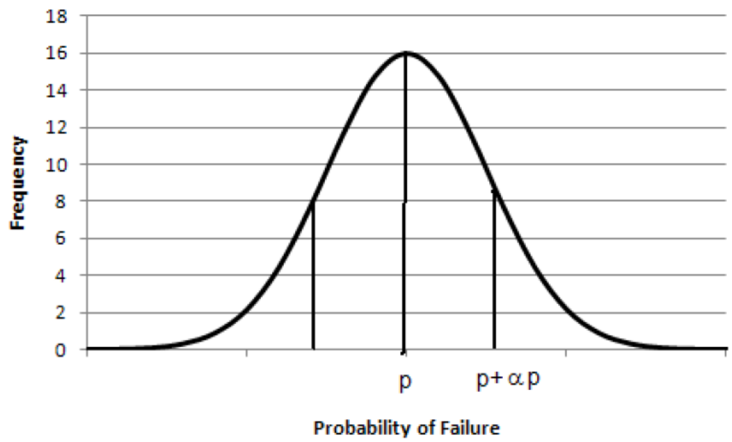

Figure 47 Distribution of Probability of Failure (Gibson, 2011).

15% of PoF was selected for mining benches (see Table 18) with a 95% confidence level and ± 10% error resulting in 2177 required number of iterations. As nowadays limit equilibrium analysis programs can calculate a large number of iterations within seconds, the default set iteration number of 10000 was used in the probabilistic analysis conducted in Liikavaara Östra.

6.4.2. Performed probabilistic analysis

In the process of probabilistic analysis, two failure criteria were used, the Mohr-Coulomb and the Barton-Bandis shear strength models. The variable input properties utilized in both failure criteria are presented in Table 20. For the joint friction angle normal distribution was utilized, similarly in (Park, et al. 2005) where 10% of the coefficient of variation was used to calculate the standard deviation of the friction angles based on the mean friction angle. For friction angles used, see Table 6 in subchapter 3.3.5. For joint orientation data, the distribution was based on Fisher distribution calculated automatically by *DIPS 7.0* software, following the same procedure as (Park, et al. 2005). For the saturated groundwater scenario, exponential distribution of water was applied as in (Pathak & Nilsen, 2003).

Table 20 Variable input properties in the performed probabilistic analysis.

	Source	Statistical distribution
Friction angle	Varying per rock type	Normal
Joint dip	From the corresponding stereonet	Fisher
Joint dip direction	From the corresponding stereonet	Fisher
Groundwater	Saturated scenario	Exponential

In the Mohr-Coulomb failure scenario, zero cohesion was assumed, while in the Barton-Bandis criterion JRC = 3 and JCS = 75 MPa were used due to the lack of data and assumptions of previous studies (Perks, 2015). Bench height of 30 m and width of 11 m were set in all domains, with varying average bench length differing by domains. Joint waviness was assumed to zero in all cases as a conservative assumption.

In both Mohr-Coulomb and Barton-Bandis criteria fully drained and fully saturated slopes were analyzed (four scenarios in total) from which the drained case with the Barton-Bandis criteria was decided to be used for final recommendations. The driving forces behind choosing the fully drained Barton-Bandis setup were the more realistic (less conservative) input values for joint surface conditions than in the Mohr-

Coulomb criteria; moreover, the comparable results with the latest rock mechanical study of the Salmijärvi pit. Both in wedge and planar failure analysis the bench design tools of *SWEDGE 6.0* (Rocscience, 2016d) and *ROCPLANE 3.0* (Rocscience, 2016c) were used, and the resulting bench face angle was selected at 15% of PoF, see Figure 48. The results of the probabilistic analysis for planar and wedge failure in drained Barton-Bandis scenario are presented in Table 21. The results of the rest of design scenarios are in Appendix 5.

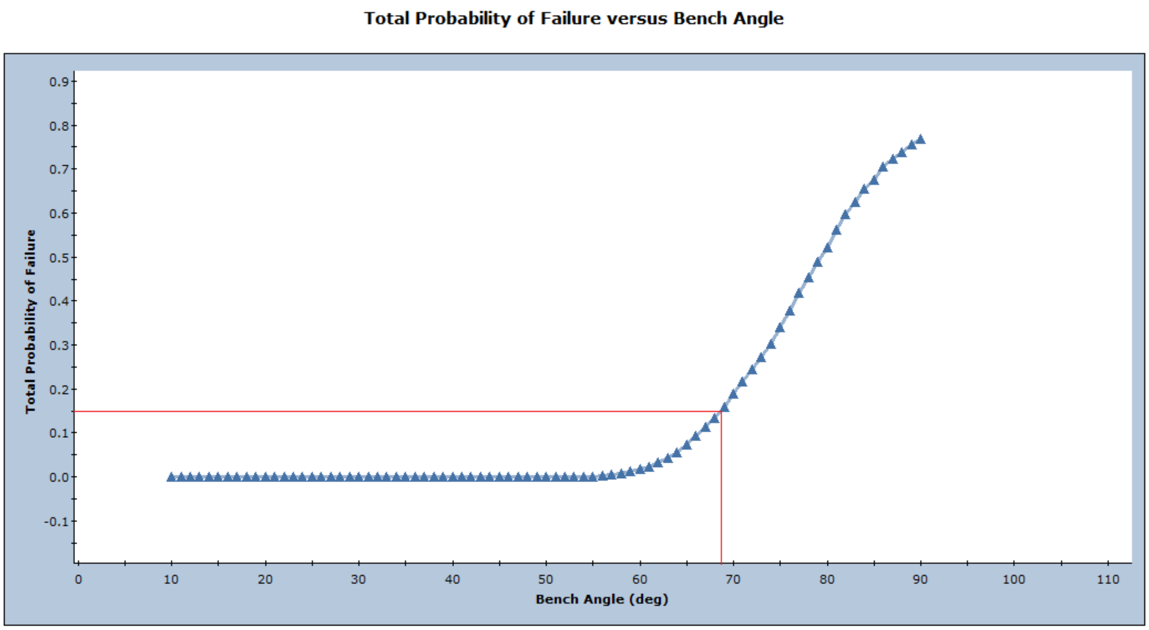

Figure 48 Probability of failure vs. bench angle plot in HW2 domain.

From the probabilistic analysis, it could be already seen that major stability issues are situated in the upper parts of the footwall domains (FW1 and FW2), where bench face angles are excessively shallow. In the domains located in the ore zone (WE1, WE2, and EE) rather steep angles deemed stable based on the analysis. In the HW1, HW2, NE1 and NE2 sectors planar failures are governing the bench face angles.

Table 21 Results of probabilistic analysis in drained, Barton-Bandis criteria setup.

Barton-Bandis			Drained				
Domain	**Drill hole**	**Bench height [m]**	**Analyzed sets of hole**	**Average slope length [m]**	**Effective bench width [m]**	**Wedge failure BFA @ 15% PoF**	**Planar failure BFA @ 15% PoF**
WE1	AITIK333	30	1-2	137	11	85	-
WE2	AITIK333	30	1-2	157	11	85	-
FW1	AITIK413	30	1-2	157	11	46	36
FW2	AITIK413	30	1-2	111	11	44	35
NE1	AITIK363	30	1-2	124	11	85	60
NE2	AITIK363	30	1-2	167	11	85	-
EE	AITIK331	30	1-2	93	11	85	-
HW1	AITIK392	30	1-4	315	11	85	67
	AITIK392	30	2-4	315	11	73	
	AITIK392	30	3-4	315	11	66	
HW2	AITIK391	30	1-4	176	11	85	66
	AITIK391	30	2-4	176	11	67	
	AITIK391	30	2-5	176	11	66	
	AITIK391	30	3-4	176	11	85	
	AITIK391	30	4-5	176	11	69	
FW1 & FW2 DEEP	AITIK369	30	1-2	157 / 111	11	65	60
NE2 lower parts	AITIK365	30	1-2	167	11	85	-
	AITIK365	30	1-4	167	11	85	-

6.5. Deterministic analysis

To check the validity of the probabilistic analysis results, a deterministic analysis was also performed for the design domains of Liikavaara Östra. The Barton-Bandis criterion with drained slope conditions was tested with the average input values and joint orientation values determined by sets in *DIPS 7.0*. In the case of toppling failure only deterministic test was conducted given the fact that both in Aitik and Salmijärvi pits, in reality, the presence of bench scale toppling is negligible, and the debris is contained on the catchment benches (Perks, 2015). The check against toppling failure in temporary domains was also decided to be disregarded. The results of the deterministic analysis are in Table 22. In the wedge stability checks the bench angles were considered stable with the wedge size under 1 MN, if the factory of safety was below 1.0, allowing minor failures to occur. In planar failure check, bench angles were determined where no planar failure was formed. All bench heights are 30 m in the conducted deterministic analysis.

Table 22 Results of deterministic analysis, Barton-Bandis scenario, drained slope case.

Domain	Drill hole	Analyzed sets of hole	Wedge failure analysis BFA [°]	FoS of wedge analysis	Size of wedge [MN]	Planar failure analysis BFA [°]	Toppling failure analysis BFA [°]
WE1	AITIK333	1-2	85	10.1	2.991	-	-
WE2	AITIK333	1-2	85	12.7	0.852	-	-
FW1	AITIK413	1-2	85	1.4	26.721	33	-
FW2	AITIK413	1-2	85	1.4	25.015	33	-
NE1	AITIK363	1-2	85	11.2	1.867	66	-
NE2	AITIK363	1-2	85	11.6	0.259	-	-
EE	AITIK331	1-2	85	5.5	1.705	-	45
HW1	AITIK392	1-4	-	-	-	73	85
	AITIK392	2-4	-	-	-		
	AITIK392	3-4	75	0.4	0.849		
HW2	AITIK391	1-4	-	-	-	75	-
	AITIK391	2-4	-	-	-		
	AITIK391	2-5	80	0.5	0.886		
	AITIK391	3-4	-	-	-		
	AITIK391	4-5	-	-	-		
FW1 & FW2 DEEP	AITIK369	1-2	70	0.5	0.082	63	-
NE2 lower parts	AITIK365	1-2	85	7.1	6.603	-	-
	AITIK365	1-4	85	-	-	-	-

6.6. Recommended bench design criteria

In the final recommended bench design criteria, the decision was based on both probabilistic and deterministic results, as well as experiences from the Aitik mine. In the following subsection, all domains are shortly described, and the final recommended bench and interramp angles are given. The final recommended bench design parameters are compiled in Table 23.

6.6.1. Ore zone domains (EE, WE1 and WE2)

All three design sectors are located mainly in the ore zone, which consists of biotite schist, andesite, and aplite. Probabilistic and deterministic wedge analysis showed that very steep bench face angles (85°) are stable; however experiences in Aitik and Salmijärvi pits indicate that similarly oriented benches situated in biotite schist are stable with 73° BFA. Planar failures were not checked as according to the planar failure criteria, slope failure do not form in these domains. Toppling failure test showed 45° face angle in the EE sector, which was decided to be disregarded. This choice was based on the fact that *ROCTOPPLE 1.0* (Rocscience, 2016e) assumes that toppling joint sets are parallel to the bench face (in the EE domain the foliation planes almost perpendicular to the bench) and minimal toppling failure existing in similar joint and bench conditions in Aitik. All in all, in these sectors 73° bench face angle was recommended based on the experience gained in Aitik and Salmijärvi.

6.6.2. Footwall side domains (FW1, FW2, NE1, and NE2)

Both deterministic and probabilistic analysis showed excessively shallow bench face angles in the upper 60 meters (2 double benches) of FW1 and FW2 domains (with granite as primary rock type). Furthermore, planar failure driven face angles at the bottom part (denoted as FW1 and FW2 DEEP, lowermost 60 m) which is situated in biotite schist. As representative data is not available from the middle part of the domains, conservatively assumed angles were recommended for the middle section. As the previously described crushed zones situated extensively in FW1 and FW2, external rock support was recommended in areas where required. With external rock support, single benching and steeper face angles were designed; thus the interramp angle of the supported areas would be same as the double benches stable at 70° BFA. Note in case of the external support scenarios (FW1-supported and FW2-supported), that the drilling offset (5.5 m) is larger than the calculated back break (2.6 m). Due to the possible circumstances (poor rock condition, rock excavation without blasting, and constructed slope faces) this deviation was disregarded.

Based on the available data and analysis, the following design criteria was recommended for the FW1 and FW2 domains:

- Single benching (15m bench height) where required, applying for external rock support with 80° BFA
- Double benching with 70° BFA where rock conditions allow (outside of the crushed zones)
- Double benching with 65° BFA (lowermost two double benches in FW1 and FW2 DEEP temporary domains)

The design criteria of FW1 and FW2 sectors are in Appendix 6. The external support methods are discussed in Chapter 8.

In NE1 domain probabilistic analysis showed 60° BFA for planar failure, while the deterministic check indicated that planar failures do not form below 66° BFA. As the input values and assumptions were rather conservative in both analysis methods (zero joint cohesion, smooth joint surface conditions, no rock bridges), the result of the deterministic planar analysis was recommended for NE1.

Although superior bench performance (85° BFA) deemed stable in the NE2 domain both deterministic and probabilistic analysis, 70° bench face angle was recommended, because in this sector, low BRQD is apparent and the presence of crushed rock areas is expected.

6.6.3. Hanging wall side domains (HW1 and HW2)

Since the hanging wall domains comprise competent rocks (good BRQD and RMR values, high UCS, no foliation, and visual inspection from borehole video filming), it is a valid assumption to recommend the steeper deterministic analysis results over the probabilistic results for bench angles in HW1 and HW2 domains.

Table 23 Final recommended bench design criteria in Liikavaara Östra.

Domain	Bench height [m]	Effective bench width [m]	Calculated back break* [m]	Total bench width [m]	Final suggested BFA [°]	Final suggested IRA [°]
WE1	30	11	9.2	20.2	73	56
WE2	30	11	9.2	20.2	73	56
FW1 - regular	30	11	10.9	21.9	70	54
FW1 - supported	15	8	2.6**	10.6	80	55
FW1 - DEEP	30	11	14	25	65	50
FW2 - regular	30	11	10.9	21.9	70	54
FW2 - supported	15	8	2.6**	10.6	80	55
FW2 - DEEP	30	11	14	25	65	50
NE1	30	11	13.4	24.4	66	51
NE2	30	11	10.9	21.9	70	54
EE	30	11	9.2	20.2	73	56
HW1	30	11	9.2	20.2	73	56
HW2	30	11	8	19	75	58
*= Drilling offset included						
**= Supported (constructed) slope case, drilling offset not applicable						

7. Overall slope stability

In contrast to bench scale failures, where the joint sets and structures are responsible for the failure; overall slope failures are mainly governed by rock mass properties, groundwater conditions and blast damage of the slope. After the kinematic, probabilistic and deterministic analysis of bench scale stability in Liikavaara Östra, the overall slope stability of designated sections was assessed. The overall stability calculations were focused on the footwall side of the pit, as the discontinuity zones are almost exclusively located in the FW1, FW2, NE1 and NE2 domains. As competent rock and stable bench conditions were assumed and calculated in the hanging wall and both pit end design sectors (HW1, HW2, WE1, WE2, and EE) the overall stability of these parts of the pit are not discussed in this report.

In the overall slope stability calculations the failure of the rock mass was assessed through the Mohr-Coulomb failure criteria. In order to obtain the large scale rock mass properties necessary for the Mohr-Coulomb model, available Hoek-Brown parameters were transformed to cohesion and friction angle values. After the model setup and data processing, simplified Janbu, and simplified Bishop limit equilibrium analysis methods were used to assess the stability of the slopes.

Three design sections were selected from the footwall side, see Figure 49. The cross sections were taken from the latest 3D model of the deposit and the planned pit, thus the position of the different rock units, discontinuity zones, and final pit walls could be represented in the best possible way. Design section denoted as “FW” is located in the FW1 domain, “Y4790” is situated in the FW2 domain, and the “NE” section is in the NE1 sector.

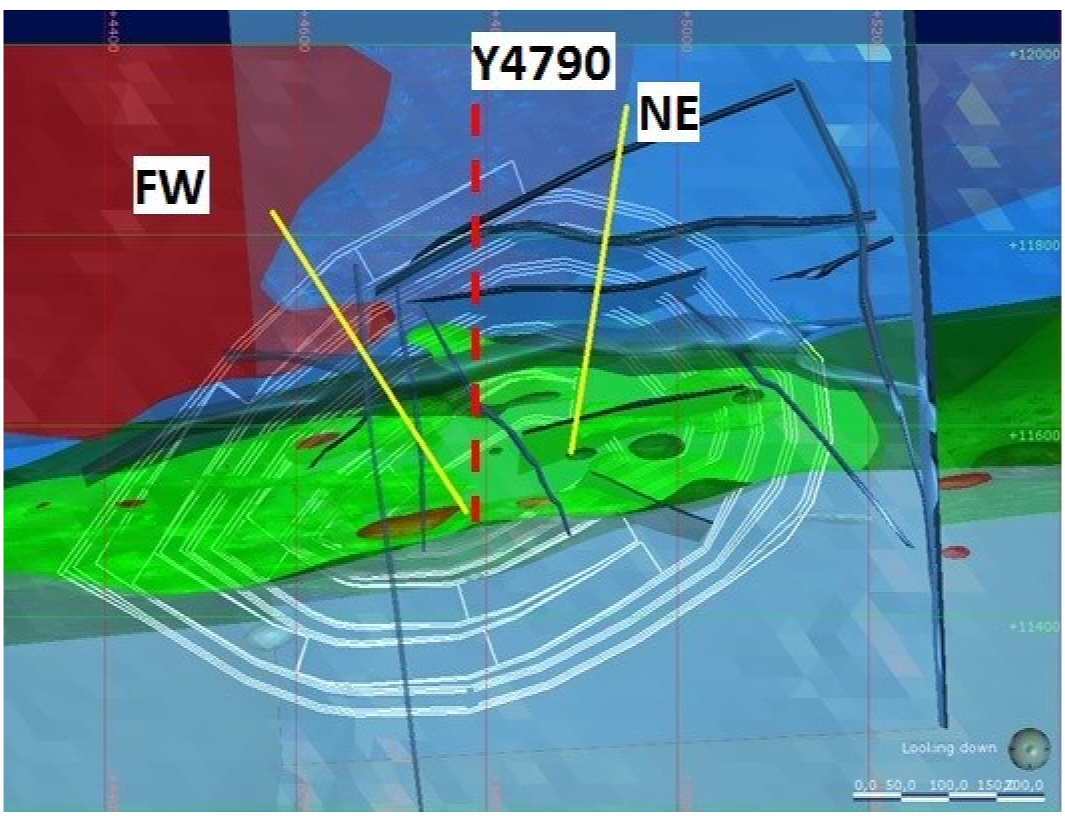

Figure 49 Location of overall slope stability design sections.

For analyzing the chosen design sections, the *SLIDE 7.0* (Rocscience, 2016a) 2D limit equilibrium analysis software was used. The position of the search grid for slip surface center points was set automatically, and grid intervals were set to the maximum of 200 grid points. In all simulations, a total number of 444 411 surfaces were checked for equilibrium.

7.1. Hydrogeological conditions

In the performed overall slope stability analysis, two groundwater scenarios were considered. In the first scenario, undrained slope conditions were assumed, while in the second groundwater scenario partial depressurization of the overall slope was analyzed. This second scenario was identical to the `Hydrogeological scenario 1` in the 2016 Aitik LOMP report (Sjöberg, et al., 2016). In this second setup, the upper two-thirds of the slope height were assumed to be drained to a horizontal distance of 100 m, while the lower third of the slope height was considered undrained, see Figure 50. This hydrogeological setting was based on the observations and dewatering practices in the Aitik main pit. There the upper parts of the slope are drained approximately 100 m from slope face, while in the benches of the lower third face seepages are observed; thus the water table is assumed to be at the slope face. The depressurization of the slope in full slope height would require extensive (and expensive) drainage measures even before the start of the mining operation, which is why it was not considered as a realistic scenario in Liikavaara Östra with the given final pit depth.

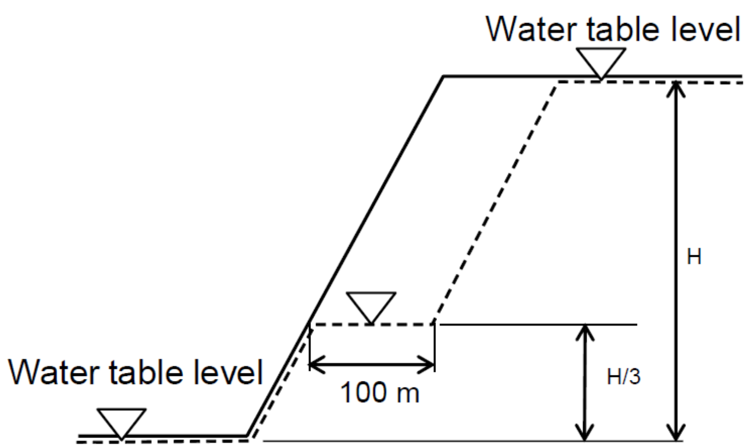

Figure 50 Groundwater scenario 2 in Liikavaara Östra (Sjöberg, et al., 2016).

7.2. Disturbance factor

In the analysis of overall stability, the value of disturbance (blast damage) factor of the rock mass (D) in the Hoek-Brown failure criterion was significantly affecting the outcome of the analysis. The factor is describing the effect of blast damage and stress relaxation of the rock mass between 0.0 and 1.0; where 0 is an undisturbed rock while 1 is very disturbed rock mass. The D-factor was first published in the updated version of the Hoek-Brown criteria in 2002 (Hoek, et al., 2002).

It was vital to apply the extent and value of factor D in a correct and reasonable way, as it could significantly underestimate (or overestimate) the stability of the rock mass. In the case of Liikavaara Östra, a conservative scenario was used, the setup of the D factor was based on the practices used in the LOMP report (Sjöberg, et al., 2016). The rock mass was assumed to be very disturbed close to the face (D=1) and moderately disturbed (D=0.7) elsewhere. The border between the two parts is displayed in Figure 51.

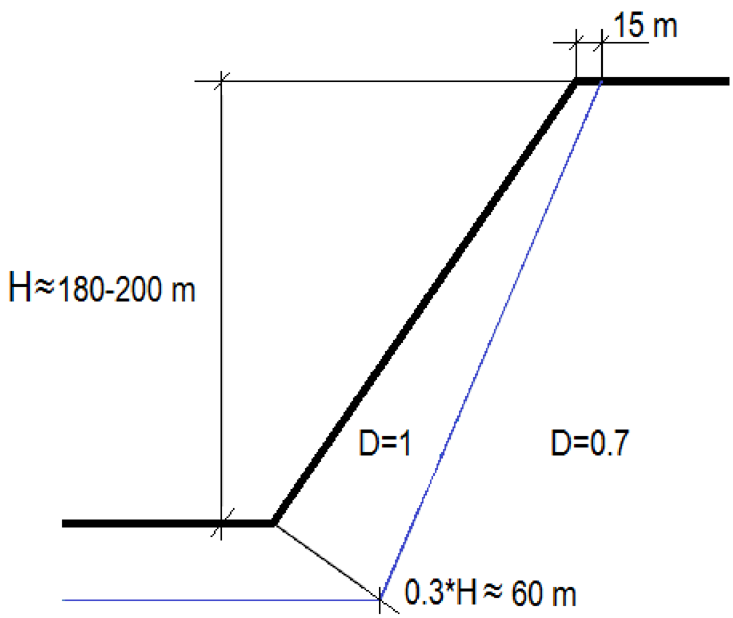

Figure 51 Design setup for disturbance factor (D) based on (Sjöberg, et al., 2016).

7.3. Rock mass properties used in overall slope stability analysis

The analysis in *SLIDE 7.0* requires the cohesion and the friction angle of the rock mass. To acquire these values for the rock types present in the slopes, the *ROCLAB 1.0* (Rocscience, 2007) software was used. In the program, the necessary values were calculated from the Hoek-Brown failure criteria to get the values required for the Mohr-Coulomb failure criteria (used in *SLIDE 7.0* program). The parameters were needed to calculate the friction angle and the cohesion of the rock units are shown in Table 24.

Table 24 Input values and its sources for friction angle and cohesion calculation.

Input parameter:	**Source/value:**
Slope height	180 m or 200 m
Unit weight	0.026 MN/m^3
Sigma C	point load tests
GSI	RMR- 5 RMR from compiled database
D	0.7 or 1
mi	Relevant values from *ROCLAB 1.0* built-in charts
MR	

The results of the calculation in *ROCLAB 1.0* for the different slope heights and disturbance factors per rock types are presented in Table 25.

Table 25 *ROCLAB 1.0* calculation results of cohesion and friction angle for different slope heights and disturbance factors.

***ROCLAB 1.0* calculation results**							
Slope height = 180 m	Hoek-Brown classification					**Calculated Mohr-Coulomb fit**	
Rock type	Sigma C [MPa]	GSI	mi	**D**	Ei [MPa]	**Cohesion [kPa]**	**ϕ [°]**
Biotite schist	62	61	10	**1**	41850	**919**	**34.3**
Crushed zone	100	20	10	**1**	12000	**216***	**14.0**
Turbidite	105	51	21	**1**	36750	**1003**	**38.8**
Granite	84	39	32	**1**	35700	**730**	**33.2**
Andesite	96	59	25	**1**	38400	**1298**	**44.2**
Slope height = 180 m	Hoek-Brown classification					**Calculated Mohr-Coulomb fit**	
Rock type	Sigma C [MPa]	GSI	mi	**D**	Ei [MPa]	**Cohesion [kPa]**	**ϕ [°]**
Biotite schist	62	61	10	**0.7**	41850	**1178**	**39.5**
Crushed zone	100	20	10	**0.7**	12000	**390***	**22.8**
Turbidite	105	51	21	**0.7**	36750	**1329**	**45.4**
Granite	84	39	32	**0.7**	35700	**1041**	**41.6**
Andesite	96	59	25	**0.7**	38400	**1638**	**49.6**
Slope height = 200 m	Hoek-Brown classification					**Calculated Mohr-Coulomb fit**	
Rock type	Sigma C [MPa]	GSI	mi	**D**	Ei [MPa]	**Cohesion [kPa]**	**ϕ [°]**
Biotite schist	62	61	10	**1**	41850	**969**	**33.5**
Crushed zone	100	20	10	**1**	12000	**229***	**13.5**
Turbidite	105	51	21	**1**	36750	**1067**	**38.0**
Granite	84	39	32	**1**	35700	**779**	**32.4**
Andesite	96	59	25	**1**	38400	**1727**	**47.3**
Slope height = 200 m	Hoek-Brown classification					**Calculated Mohr-Coulomb fit**	
Rock type	Sigma C [MPa]	GSI	mi	**D**	Ei [MPa]	**Cohesion [kPa]**	**ϕ [°]**
Biotite schist	62	61	10	**0.7**	41850	**1241**	**38.7**
Crushed zone	100	20	10	**0.7**	12000	**415***	**22.2**
Turbidite	105	51	21	**0.7**	36750	**1416**	**44.6**
Granite	84	39	32	**0.7**	35700	**1113**	**40.8**
Andesite	96	59	25	**0.7**	38400	**2184**	**52.4**

*Cohesion = 0 kPa is assumed for discontinuity zones

After the geology, pit dimensions, rock mass properties and hydrogeology were set in the models; calculations were executed. First the overall slope angle of the actual pit design was investigated for undrained and drained scenarios based on the cross sections of the pit design. Following this, the resultant overall slope angle was tested for undrained and drained conditions. The difference between the actual and the resultant slope angles was due to the small final pit depth. The resultant slope angle was calculated with 200 m slope height, one ramp width (40 m) and the interramp angle. As more than one ramp was located within the maximum considered interramp height (200 m) in Liikavaara Östra, the actual OSA of the pit design was less steep than the resultant OSA.

The results of the overall slope analysis are in Table 26. Figure 52 is an example of potential large scale failure in the NE design section for the undrained scenario. The rest of the figures of the design sections and all design scenarios are shown in Appendix 7

Table 26 Results of the overall slope stability analysis in Liikavaara Östra.

Design section	OSA checked [°]	Hydrogeology	Extent of slip surfaces below FoS = 1.3	Overall stability
NE	36	undrained	Deep, large scale failures	NOT ACCEPTABLE
NE	36	drained	Shallow, in crushed zones only	ACCEPTABLE
NE	47	undrained	Deep, large scale failures	NOT ACCEPTABLE
NE	47	drained	Shallow, in crushed zones only	ACCEPTABLE
FW	37	undrained	Shallow, in crushed zones only	ACCEPTABLE
FW	37	drained	Shallow, in crushed zones only	ACCEPTABLE
FW	46	undrained	Deep, large scale failures	NOT ACCEPTABLE
FW	46	drained	One interramp slip surface at FoS=1.29	ACCEPTABLE
Y4790	38	undrained	Shallow, in crushed zones only	ACCEPTABLE
Y4790	38	drained	Shallow, in crushed zones only	ACCEPTABLE
Y4790	45	undrained	Shallow, in crushed zones only	ACCEPTABLE
Y4790	45	drained	Shallow, in crushed zones only	ACCEPTABLE

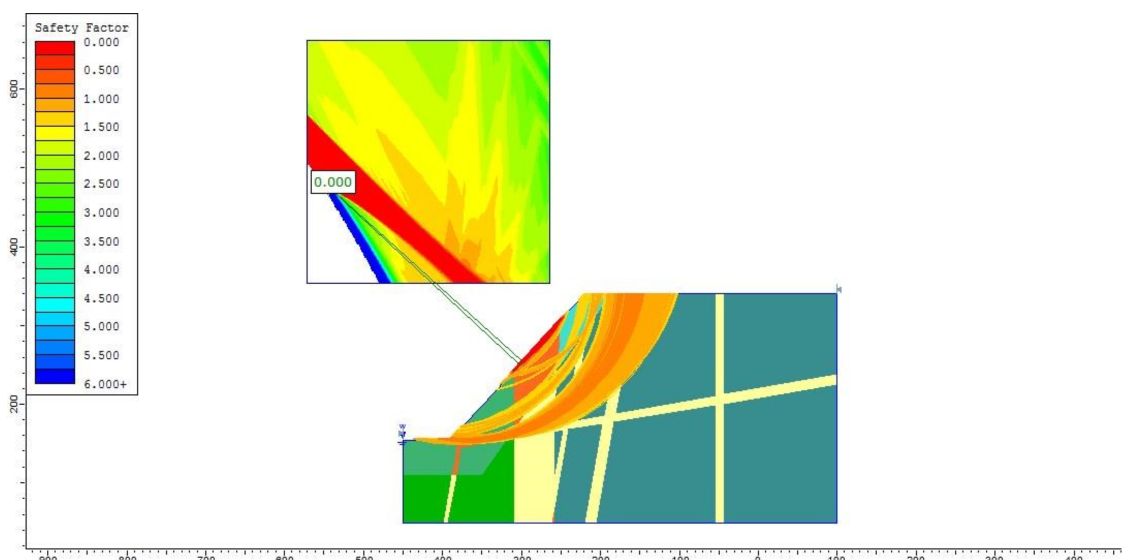

Figure 52 Example for large scale failure in the NE design section, undrained slope and resultant OSA 47 °. All failure surfaces shown are below 1.3 FoS.

7.4. Recommended overall slope angles for Liikavaara Östra

Based on the limit equilibrium analysis of the selected design sections and assumed scenarios, the recommended overall slope angles for Liikavaara Östra are the following, see Table 27. As mentioned previously the resultant OSAs were calculated for 200 m interramp height, 40 m ramp width and the interramp angle. In domains where no overall slope stability check was executed, the resultant OSA was recommended; while in the footwall side domains the angles were based on either the resultant OSA or the tested results. In FW1 and FW2 sectors the resultant OSA was 46°, which for drained conditions showed one interramp size failure surface (100 m vertical height) at 1.29 FoS. Considering the conservative input properties of the model, and the vicinity of the FoS value to the threshold limit of 1.3, the resultant OSA of 46° was suggested in the FW1 and FW2 sectors. In the case of NE1 domain, the tested OSA was stable at 47° (2° steeper than the resultant OSA), but due to the possibility of planar failure at bench scale, increasing the OSA from 45° to 47° was not recommended in this domain. In the NE2 domain, the tested OSA 47° was considered to be stable based on the analysis.

Table 27 Recommended overall slope angles for Liikavaara Östra.

Domain	Resultant OSA [°]	Tested OSA with depressurized slope [°]	Recommended OSA [°]
WE1	49	-	49
WE2	49	-	49
FW1	46	46	46
FW2	46	46	46
NE1	45	47	45
NE2	47	47	47
EE	49	-	49
HW1	49	-	49
HW2	50	-	50

From the results of the overall slope stability check presented in Table 26, it can be seen that the slopes are stable in depressurized hydrogeological conditions with both actual and resultant overall slope angles as well. However, for the undrained cases the NE section fails with both actual and resultant OSAs, and in the FW section overall fail slip surfaces occur in the resultant OSA setup. The Y4790 section seemed stable in both OSA configuration and hydrogeological scenario according to the models. Based on these results it is strongly recommended that drainage of the slopes is executed in Liikavaara Östra to achieve acceptable depressurization levels. It is also advised to continuously drain the benches from the start of the operation; therefore enough time would be available for successful depressurization. Detailed drainage methods are not discussed in this thesis as the topic is out of the scope of this study.

Although the overall slope stability was considered acceptable in the depressurized cases, in all design sections and scenarios shallow bench scale slip surfaces were formed. The failures were limited in size to 30-60 m height and their location were solely in the discontinuity zone outcroppings. As mentioned before, these crushed zones have poor rock conditions, raising concerns regarding slope stability. According to the performed *SLIDE 7.0* analysis, these failures do not affect overall slope stability (the overall slip surfaces occur in undrained hydrogeological conditions), but potentially causing smaller bench scale or interramp scale raveling type failures.

In non-final benches, production areas, these failures and the resulting bench angles of the natural angle of repose are acceptable; with single benching and wider effective bench widths larger failures can be avoided or controlled. If required, cheap waste rock buttresses can be also used as external support enabling slightly steeper benches than the natural angle. However, it is not advised to install permanent facilities (ramp, switchback, dewatering station, etc.) in or near these rock conditions, but if it is inevitable, external rock support measures are recommended to ensure safe and uninterrupted operation. The design process and cost estimation for external rock support methods are subject to Chapter 8.

8. External support

The application of external support was investigated in Liikavaara Östra for multiple reasons. In the domains closely located to the E10 road (FW1 and FW2), the shallow bench angles required for stable conditions imply that the relocation of the road would be inevitable, resulting in major expenses in the range of 100-200 million SEK. Moreover, as a result of the shallow bench angles, increased stripping ratio would further reduce the profitability of the operation. Installation of permanent objects, such as a ramp, dewatering station, electric transformers, etc. would also require a stable environment. These abovementioned reasons motivated to investigate the possibility of application external rock support, to maintain the desired bench and interramp angles, as well as safe and uninterrupted mining operation.

Based on the overall slope stability check, stability issues are not only exclusive to the FW1 and FW2 domains but in the whole footwall side can be expected where the previously described crushed zones outcrop in the pit. In these areas, circular slip failures can develop in the height of 30-60 m according to the design sections tested in *SLIDE 7.0* in chapter 7. For this reason, multiple external support methods were considered and designed to provide a solution for these stability issues.

It is also important to consider the behavior of the exposed crushed zones. The material either can be analyzed as a rock slope, or based on the high in-situ fragmentation it can be evaluated as a soil slope. In the case of soil like (cohesionless granular material) behavior, support methods designed for rock mass are not stable; thus different approach must be examined for this scenario.

External support methods are briefly described and compared here for both soil and rock mass behaving slope scenarios. Moreover, preliminary cost estimation is provided, based on which methods for support are advised for slope stabilization in the crushed rock zones of Liikavaara Östra.

8.1. Design of external support methods

8.1.1. Support method for soil slope scenario

For soil behaving scenario, a gabion retention wall system was considered and designed as a support method. Although gabion wall systems are widely used in civil engineering projects as a proven and efficient method for soil slope stabilization, the application of the method in the mining sector is limited. Apart from applications for main crusher establishment, stockpile containment structures and as ramp reconstruction (Maccaferri, 2016), it is not frequently used as an extensive slope support technique in open pit mines.

In Liikavaara Östra several reasons motivated the consideration of using gabion cages as slope support. As the climate conditions are harsh in Northern Norrbotten (long winters, seven months with below zero average temperature (Climate Data, 2016)) gabion wall systems have a significant advantage that it can be constructed in temperatures below zero, thus giving more flexibility in the construction period. Gabion walls can also bear high loads, and can be applied as a support of ramps as it was implemented in the Mokopane Platinum mine in Zambia, see Figure 53. Gabion cage system is also cheaper than preformed concrete cantilever systems. Another advantage for the gabion cage is that available site material (crushed and screened waste rock) for filling the cages can decrease the overall cost of application.

Figure 53 Ramp stabilization in Mokopane, Zambia. The maximum height of retaining structure is 15m, total length 150 m (Maccaferri, 2016).

Gabion wall calculations were conducted with the student license of *GEO5* (Fine, 2016) civil engineering software program. First 7.5 m high vertical bench (half of a 15 m single bench) was designed and checked against slip, overturning, horizontal pressures and forces acting on the mesh material. The designed gabion wall is displayed in Figure 54. Afterward, the bench configuration of stacked gabions was tested for a hypothetical 60 m high and 50 m wide crushed zone outcrop situated in granite as shown in Figure 55. The bench face angle was 80° for 15 m high benches and the interramp angle was 54° for the whole gabion supported bench. The applied calculation method was Bishop method of slices, and the input rock mass properties were taken from the overall slope stability check (D=0.7 case, 180 m high slope input scenario, see Table 25). The slope was considered fully drained. The tested slope was found stable with a factor of safety of 2.09. This is judged to be adequate, taking into account the limitations of the calculation method and the simplification of the model. Detailed screenshots from the design program are presented in Appendix 8.

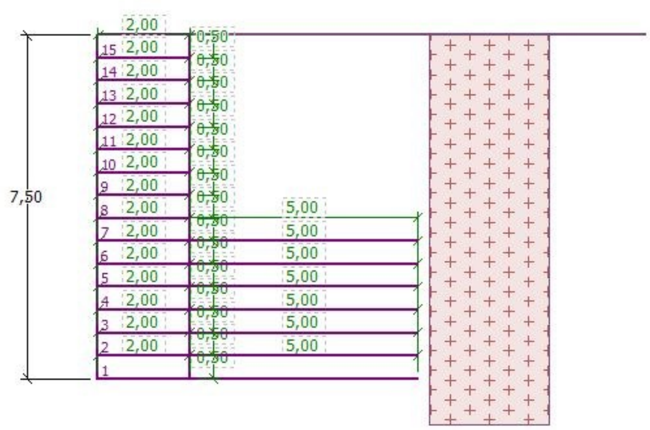

Figure 54 Gabion wall design in Liikavaara Östra.
Gabion cage size: 2m X 1m X 0.5m. Mesh extension is applied in 5 m length in the lower 3 m section in every 0.5 m.

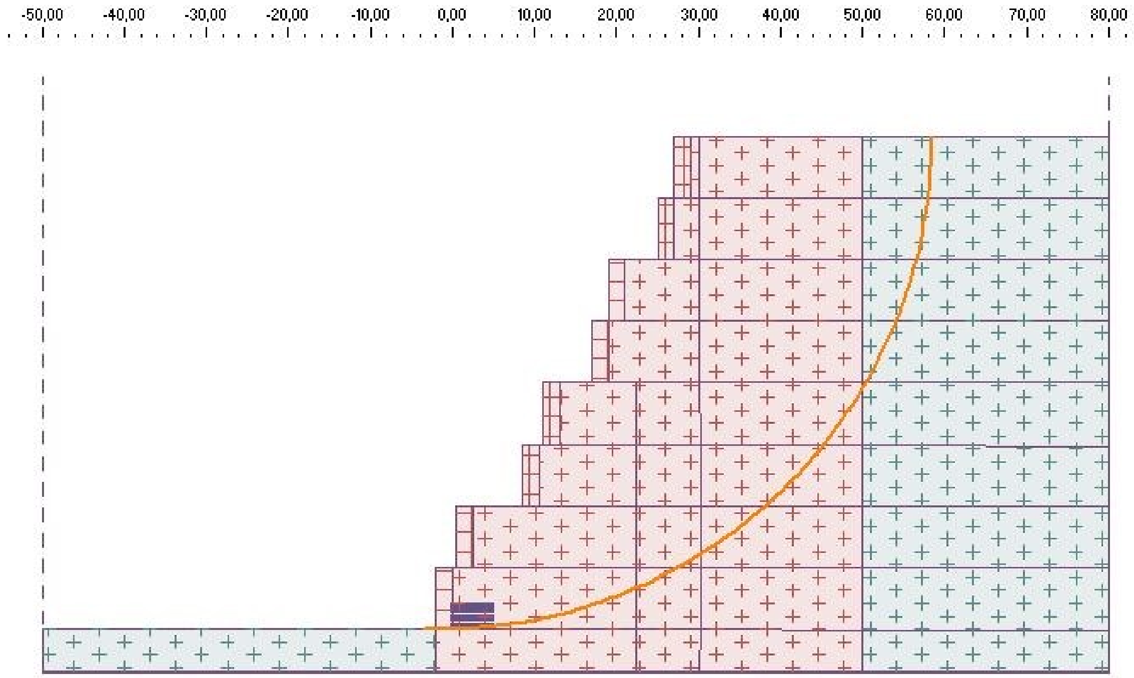

Figure 55 Bench design model with gabion cage system for hypothetical crushed rock outcrop.

8.1.2. Support method for rock slope scenario

For the rock slope scenario, multiple support methods were considered, such as shotcreting, meshing, bolting, as well as the combination of these practices. Based on discussions, experience and the state of the broken rock mass, single support methods were eliminated early from the design process. Meshing and bolting, was not decided to be recommended as the anchoring of bolts would be problematic in poor, crushed rock; thus the mesh would suffer movements and would not hold back the slope from failure, only it would slow down the process.

As sufficient support methods the combination of shotcrete and bolts, and shotcrete layer with mesh and bolts were considered. The application of shotcrete does not allow the first block (key block) to fall out resulting in progressive failure, while the systematic bolting holds together the rock blocks behind the shotcrete cover. Meshing would allow some movement of the system, if cracks would form in the shotcrete layer due to tension or blast damage. Fiber reinforced shotcrete can also be utilized as an alternative to mesh in order to avoid crack formation in the shotcrete cover. Major disadvantage of both methods that the shotcrete layer limits the application to a short period of the year, when temperatures are above 0 °C. The proposed support methods are similar to the practice applied in schistose rock slope displayed in Figure 56.

Figure 56 Rock slope support used in schistose rock slope in Venezuela. The slope is supported with shotcrete cover, wire mesh, rock bolts and anchored cables (Goodman, 1980).

The support method for rock slope scenario was designed in *SWEDGE 6.0*. According to the design software calculations, application of shotcrete in the thickness of 0.1 m and 1 MPa tensile strength would singlehandedly hold back the broken material in a 15m high bench, see Appendix 8. As mentioned previously, the application of bolting, or bolting and meshing were considered to be added to the support method to limit the movements of the rock mass to an acceptable level. The pattern advised for bolting is 3 m X 3 m with 5 m long grouted rebars, which is a similar system to the bolting pattern used in the Maurliden open pit of Boldien. In Maurliden systematic bolting is utilized to stabilize the foliated rocks in the pit walls (Boliden, 2016b). The recommended support for a single bench is displayed in Figure 57.

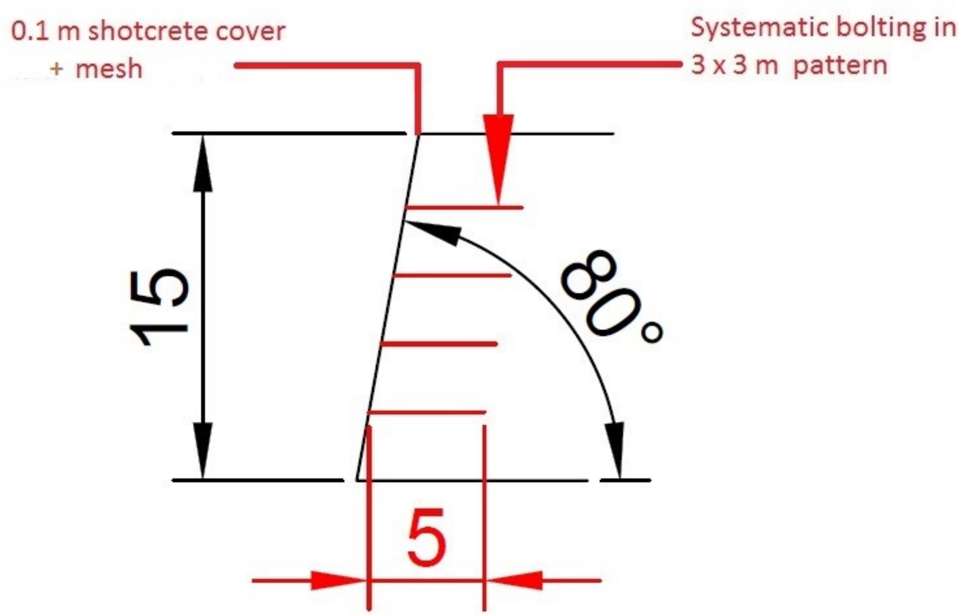

Figure 57 External support design with shotcrete, welded mesh and systematic bolting.

8.2. Cost estimation of external support methods

8.2.1. Permanent slope support

For all the support methods considered, the preliminary cost of the application was estimated based on general cost estimation of material, machine and man-hour costs, see Table 28. The cost calculations of support methods are in Appendix 9.

Table 28 Cost estimation comparison of external support methods for Liikavaara Östra.

	Cost per bench face area supported [SEK/m^2]	Cost per meter of supported bench [SEK/m]	Scenario
Gabion wall	1 382	21 000	Soil scenario
Shotcrete + mesh + bolts	201	3 050	Rock mass scenario
Shotcrete + bolts	147	2 242	Rock mass scenario
Mesh + bolts	87	1 325	Rock mass scenario

It is apparent from Table 28 that it is significantly cheaper if the supported mass is considered as rock mass instead of soil. For the rock mass case advised support systems are at least seven times less expensive than the support recommended for the soil scenario. Although the rock mass scenario methods are considerably cheaper than the support method for the soil behaving slope, financial considerations should not motive the selection of the more economical solution over safety. As the rock mass scenario designs do not take into account the in-situ fragmentation of the rock (only the orientation of the joints), major underestimation of slope stability can arise. Based on the currently existing information (extensive sections of core with zero BRQD, geophysical measurements) it can be assumed that in some areas the supported mass should be considered as soil behaving material, in which case the gabion wall design is recommended as external support method for final benches. The systems designed for the rock mass scenario are advised to be used wherever possible (in less crushed areas, where BRQD is somewhat better), thus the cost of external support can be decreased.

8.2.2. Temporary slope support

For non-final benches, however, it is not recommended to apply the abovementioned external support methods for financial considerations, unless it is clearly necessary due to safety reasons. If crushed zone is exposed in mining benches, there are two options available. The first option is to let the benches fail and form the naturally stable bench face angles (BFA 35°-45°). Even if this would temporarily limit ore blocks from excavation, it can serve as a cheap alternative to permanent external support methods. The second alternative for temporary slope support is to use compacted rockfill buttresses. With the application of buttresses, steeper bench angles can be achieved than the natural slope angle of heavily fragmented rock slopes. According to the case study conducted in the Gruvberget open pit of LKAB, with compacted rockfill buttress 56° stable BFA can be achieved at a relatively low construction cost and time (Bergman, 2016). The application of buttresses would result in less ore loss and above a certain limit of exposed ore, it is economically more feasible than operating with natural bench angles as more ore can be extracted.

8.2.3. Cost estimation scenario for the Footwall domains

In the previous subchapters, the different types of external support methods were described and compared to each other, resulting in an application cost per meter of supported bench. In the following, a cost estimation scenario is presented for the upper segment of FW1 and FW2 domains. In this scenario it was assumed that the upper 60 m of both domains were stable at 36° BFA, at steeper bench angles external support was required to maintain safe benches. First, as a reference case, it was assumed that the 15 m high (single) benches were stable at 80° BFA, mining of excessive waste material was calculated from this profile. The approximate cost of relocating the E10 road was also incorporated in the scenario. Afterwards, the total application costs of different support methods were calculated and compared, see Table 29. The detailed calculation tables are in Appendix 9.

Table 29 Cost estimation case for FW1 and FW2 domains.

	Cost of additional waste mining + road relocation [SEK]	Cost of applied support [SEK]	Grand total [SEK]
Stable @ BFA 80° (reference case)	-	-	-
Stable @ BFA 36°	38 528 350	-	38 528 350
Stable @ BFA 36° + E10 relocation	138 528 350	-	138 528 350
Gabion retention wall	-	22 512 000	22 512 000
Shotcrete (10cm) + mesh + bolts	-	3 269 298	3 269 298
Shotcrete (10cm) + bolts	-	2 403 281	2 403 281
Mesh + bolts	-	1 419 884	1 419 884

Table 29 clearly presents that in the examined case of the Footwall domains, the application of external rock support is more feasible than excavating additional waste rock material. With including the relocation cost of the E10 road, it is evident that the utilization of support methods can significantly decrease the investment expenses of the project. Based on this estimated scenario the application of external rock support is strongly advised in the FW1 and FW2 domains; thus the relocation of the E10 road can be avoided, and steep mining benches can be maintained, resulting in better ore recovery (better stripping ratio) and decreased investment costs. This is true even if the rock quality is very low ("soil scenario") and the (comparatively) more expensive gabion walls are implemented.

9. Discussions

9.1. Data collection and analysis discussion

During the data collection and analysis phase of the thesis project, certain limitations of the data acquiring and processing methods were exposed.

Point load testing is a fast but inaccurate field testing method for obtaining compressive strength data. A huge advantage of the process that in a short period of time large number of tests can be conducted, with no or minimal sample preparation. On the other hand, the number of invalid tests is substantial, and the accuracy of the method is lower than the UCS test method`s. During this year´s point load testing campaign 358 samples were tested, from which 251 tests were valid. The scatter of the results was rather large which was due to the natural scatter of the rock strength and the accuracy of the point load testing method. The scatter of the results was exceptionally large in rock units where the number of samples was limited. Especially in the Lina granite (11 valid samples) high standard deviation was measured with 75% coefficient of variation. As no UCS measurements have been conducted in Liikavaara Östra, the conversion factor of Aitik was used to obtain UCS data from the point load index results. Without the rock type specific UCS test correlations, it is rather difficult to determine the accuracy of the point load testing. With UCS sampling not only more precise rock strength results can be obtained, but the conversion factor can be calculated for the different rock types in Liikavaara Östra. However, the UCS testing of the foliated rock units (such as biotite schist, and Lina granite) will be rather problematic, resulting in uncertainties in their correlation factors.

The collection and analysis of joint orientation data were complicated in the footwall side of the pit. While the video filming and interpretation of boreholes AITIK391 and AITIK392 was trouble-free, in the footwall side borehole filming was not possible due to extremely poor hole conditions. Only one partially oriented borehole was available, from which 24 discontinuities has been mapped. Although the number of structures was not representative and potential errors can arise from the utilization of this information, as no other joint orientation data was available from the domains, this dataset had to be used to design the upper segment of FW1 and FW2 domains. With further oriented core drilling in the area it is anticipated to gain more joint orientation data, and hence more reliable slope design can be achieved in the future.

From the available discontinuity orientation data, joint sets were determined in *DIPS 7.0* using several steps. First, the automatic clustering option was used with 30° set window. Afterward the clustered sets were further refined. Where it was possible automatic set windows were used, while to determine (sub)-horizontal sets, the free-hand set marking option was used. Finally, with additional personal discussions, the effect of one´s personal interpretation was aimed to be limited. With the processing of joint data in steps, the accuracy of joint sets deemed satisfactory, on which further analysis could be based.

9.2. Bench slope design discussion

During the bench slope design process both probabilistic and deterministic analysis methods were calculated. In the final recommendations, however, not only these results but the experience gained in Aitik were also utilized to determine the bench face angles. The reason behind using the results and experience in combination was to screen out certain limitations of the design programs (*SWEDGE 6.0* and *ROCPLANE 3.0)* and design methods (probabilistic and deterministic). Both software did not take into account the frequency of the joints (BRQD), thus serious under- or overestimation of bench angles could arise. In the Hanging wall domains (HW1 and HW2) the spacing of joint sets was rather large, resulting in potential underestimation of bench angles in these sectors. However, in the EE, WE1, WE2 and NE2 domains the bench angles were most probably overestimated. In the East and West pit end sectors both methods showed 85° stable BFA while in Aitik in similar circumstances only 73° BFA can be maintained. Since the main rock type in the slopes is biotite schist with strong foliation and high discontinuity frequency, it was reasonable to assume that the models overestimated the bench performance, and the slopes in Liikavaara Östra will likely to behave in a similar way like in Aitik. However further investigations in these domains are vital, because steeper bench angles in the ore zone domains directly result in better ore recovery. Moreover, in the NE2 domain the slopes were found to be stable at 85° BFA, while in the sector low BRQD of the rock mass and presence of crushed zones is expected. For this reason, BFA 70° was estimated there as a conservative assumption. Similarly to the ore zone sectors, in the NE2 domain further data collection and modelling can confirm or disprove the currently recommended parameters.

In the rest of the sectors the deterministic results were favored over the probabilistic findings, increasing the bench angles by 5°-10°. This increase and favoring the deterministic results was based on several aspects. Joint persistence is still unknown in Liikavaara Östra as only drill hole data is available from the area. In the design process as a worst case scenario long and persistent joints were assumed, with zero cohesion and the absence of rock bridges; if in the models minimal cohesion is assumed, thus rock bridges are "simulated" in the model the stability of slopes increase significantly. The joint waviness was also set to zero assuming worst case scenario, while in previous studies a value of 7.5 was used. Although joint waviness is not known it can be anticipated that it will be larger than zero. JRC and JCS values were also conservative in both design methods, further decreasing the stable bench angles. All these assumptions and input values make both probabilistic and deterministic design rather conservative. It is evident that probabilistic results give more moderate final results as not only the mean values are considered in the analysis, but numerous simulations (to model scatter) are tested. Nevertheless, if the conservative input values, and the differences between the design methods are considered, moreover minor failures on the benches are accepted, deterministic methods can be used as reasonable and realistic final design recommendations. In future studies when more joint surface related data is available, it is recommended to favor the probabilistic analysis over the deterministic approach in the design process, as more realistic results can be obtained by the former method.

9.3. Overall slope stability discussion

The overall slope stability check was conducted by limit equilibrium methods in *SLIDE 7.0.* Although the applied methods and models were considered satisfactory for this preliminary design stage, the limitations of the design process and the obtained results indicate that a numerical modeling stability check is recommended in the upcoming studies. The simplified Janbu method satisfies the vertical and horizontal force equilibrium, but ignores the shear forces between the investigated slices. The simplified Bishop method only calculates the vertical equilibrium and the overall moment equilibrium, horizontal forces and shear forces are also ignored in this approach. Furthermore, stability is only checked at arbitrarily chosen slip surfaces in both practices. Also, the limited information regarding the precise location of the crushed zones, the total absence of hydrogeological data and preliminary rock mass input data further reduce the accuracy of the models. These uncertainties and the potential overall slip surfaces indicated in undrained conditions imply further investigations to be undertaken, applying numerical modeling to minimize the limitations given by the calculation methods and the geological model.

9.4. External support discussion

During the design of the external rock support for the crushed zones, a major question was whether considering the supported mass as soil or rock. As the core samples indicated long sections with zero BRQD, core loss and even sand-sized material, the hypothesis to design the support need for cohesionless material was a valid assumption. Nonetheless, design criteria for rock mass behaving slopes were also calculated, which results can be utilized in slopes where conditions are somewhat better than entirely crushed areas. To distinguish the slope behavior is fundamental; apart from safety considerations, significant financial differences exist between the support methods for soil and rock slopes. That is why in future studies two main aspects should be analyzed, namely detailed design and cost analysis of support methods for soil and rock slopes, and precise location of areas where soil retention system and rock slope support practices should be used.

10. Conclusions

Based on the data collection campaign undertaken in 2016, the rock mechanical knowledge in Liikavaara Östra has significantly increased. The core logging and point load testing results concurred with the previous studies and gave further information regarding the joint surface conditions, joint filling, and intact rock strength. The joint orientation data obtained this year verified the already determined main joint sets, but also detected a previously suspected set. This joint set was previously under-represented due to the bias caused by the orientation of boreholes drilled before 2016. With newly acquired joint orientation data, domain specific design of the slopes became possible.

The rock mass in Liikavaara Östra proved to be in slightly worse condition than in Aitik and Salmijärvi. BRQD and RMR values are generally lower in Liikavaara Östra, while the compressive strength of rocks is broadly in the same range. The compiled joint orientation data of Liikavaara Östra show similarities with the two producing pits. High angle foliation, horizontal bedding, and N-S trending sets can be recognized in Liikavaara Östra as well; however certain differences appear when stereoplots are analyzed individually. The joint set caused by the Lina granite intrusion also differs from the discontinuity sets in Aitik and Salmijärvi. In the previously detected crushed zones, rock conditions and slope angles are expected to be significantly worse than in Aitik. With the recent drilling program, the already found zones were confirmed, and further sections of broken rock were found.

The design criteria based on the compiled new and historical data showed that bench and interramp angles can be increased compared to the previous design criteria. However, the new parameters require minimized back break and blast damage achieved via improved blasting practices, such as presplitting and/or smooth blasting. In spite of the poor rock conditions, steep bench face angles can be obtained in the exposed crushed rock slopes using external rock support practices. With the application of gabion wall or meshed and bolted shotcrete wall in the crushed zones, a major increase in bench face and interramp angles can be achieved; moreover the relocation of the E10 road can be avoided.

Based on the limit equilibrium models, the overall stability of footwall slopes assuming partially undrained conditions were proved to be stable with the current design. The hanging wall and ore zone sectors were considered to be stable with the resultant overall slope angles. The final recommendation for the bench, interramp, and overall angles as well as bench heights and catch bench widths are given in Table 30. The final position of domains is displayed in Figure 58.

Table 30 Suggested rock mechanical design criteria for Liikavaara Östra.

Domain	Bench height [m]	Effective bench width [m]	Calculated back break* [m]	Total bench width [m]	Final suggested BFA [°]	Final suggested IRA [°]	Final suggested OSA [°]
WE1	30	11	9.2	20.2	73	56	49
WE2	30	11	9.2	20.2	73	56	49
FW1 - regular	30	11	10.9	21.9	70	54	46
FW1 - supported	15	8	2.6**	10.6	80	55	-
FW1 - DEEP	30	11	14	25	65	50	-
FW2 - regular	30	11	10.9	21.9	70	54	46
FW2 - supported	15	8	2.6**	10.6	80	55	-
FW2 - DEEP	30	11	14	25	65	50	-
NE1	30	11	13.4	24.4	66	51	45
NE2	30	11	10.9	21.9	70	54	47
EE	30	11	9.2	20.2	73	56	49
HW1	30	11	9.2	20.2	73	56	49
HW2	30	11	8	19	75	58	50
*= Including drilling offset							
**= Supported (constructed) slope case, drilling offset not applicable							

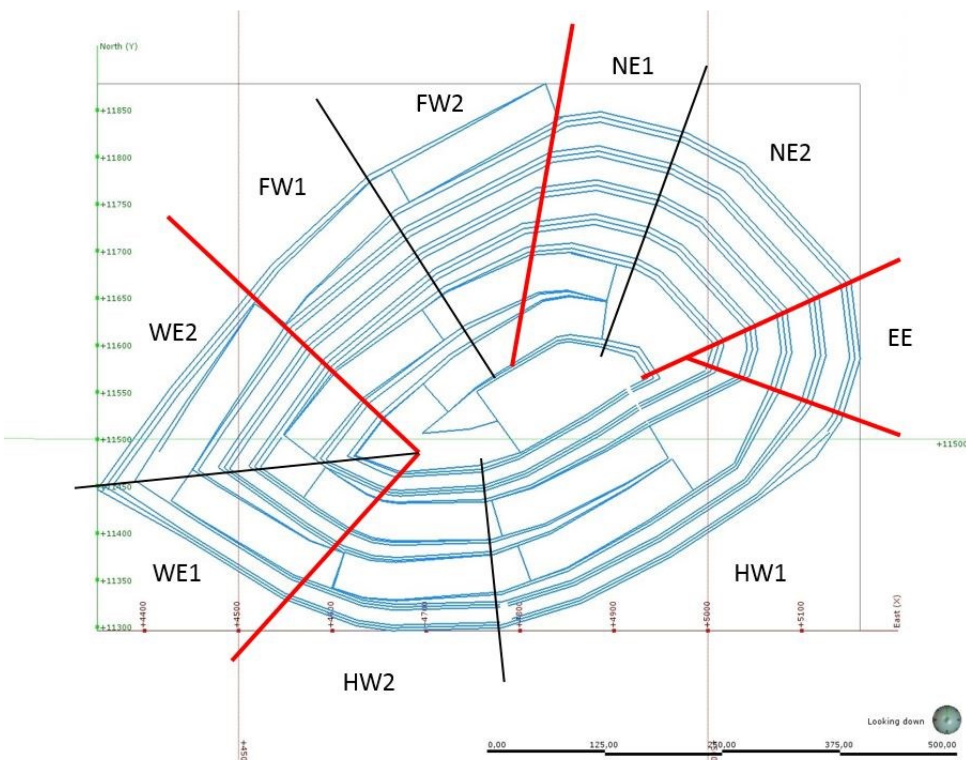

Figure 58 Final position of design domains in Liikavaara Östra.

11. Recommendations

Based on the findings of this thesis the following recommendations are advised for further investigations and mine operation:

- It is strongly advised to drill rock mechanical holes behind the final pit limit of the footwall side, to obtain joint orientation data from the area. Due to the poor rock conditions, it is also recommended to orient the core; thus joint orientation data would be still available, even if borehole video filming is not possible because of the poor borehole conditions. It is also advised to drill the holes in a similar direction as the AITIK391 and AITIK392 holes, verifying the joint sets determined in 2016. In Figure 59 the proposed boreholes are displayed. Further information regarding the rock mechanical holes are in Appendix 10.

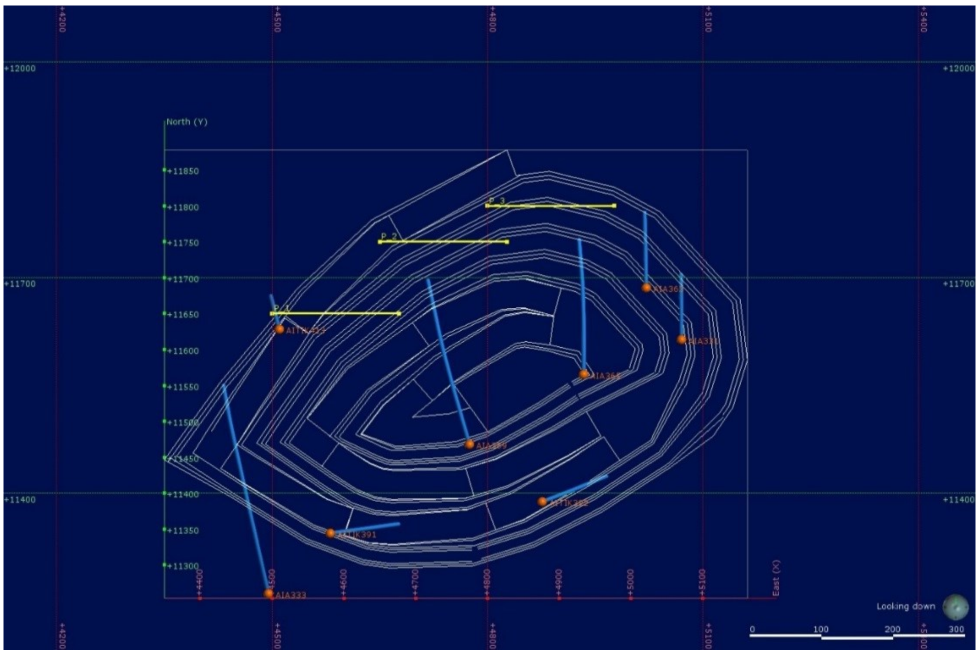

Figure 59 Top view of proposed boreholes in Liikavaara Östra. Yellow traces display the proposed drill holes, light blue traces indicate the existing holes with joint orientation information.

- During the subsequent geological drilling program, in boreholes near the final pit limit, cores are recommended to be logged according to the company standard rock mechanical logging system to gain more information regarding joint surface and filling properties.

- As UCS measurements were not executed to this date in Liikavaara Östra, compressive strength data was obtained from point load index results, and the correlation factor calibrated in Aitik. For this reason, it is recommended to systematically sample the UCS of all rock types in Liikavaara Östra. Determining elastic properties would also be beneficial, which findings could be used in overall slope stability investigations in the future. Determining the correlation factor between UCS and point load index would also help to obtain more precise data from point load tests conducted in the future.

- Due to the lack of joint shear strength data, several assumptions had to be made in the design process in Liikavaara Östra, decreasing the accuracy of design. As rock types differ from the lithology of Aitik, it is advised to measure the following joint properties: friction angle, joint cohesion, JRC, and JCS. To obtain data for the mentioned attributes, tilt tests on mated joint samples (friction angle), shear tests of discontinuities (joint cohesion), profilometer (Barton´s comb) measurements of joint surfaces (JRC), and Schmidt hammer test of joint surfaces (JCS) are recommended.

- Once mining has commenced, photogrammetry based bench mapping is recommended, to verify the joint sets determined from borehole information. Mapping should first focus on the footwall side domains.

- The overall slope stability check in Liikavaara Östra showed that the overall stability is close to its limits. As both the models and the performed limit equilibrium analysis had limitations and simplifications, the results should be treated with some caution. That is why numerical modeling of the overall slope stability is recommended, especially for the footwall side of the mine.

- Hydrogeological conditions are unknown in Liikavaara Östra as no studies have been performed to present day. As slope stability is sensitive to groundwater, it is strongly recommended to conduct detailed hydrogeological studies in the area.

- During the overall slope stability check, it appeared that high groundwater levels in the slope significantly affect the stability. If the conducted hydrogeological studies indicate the presence of groundwater, drilling of dewatering holes is advised, to prevent large scale failures. With the boring of (sub)-horizontal holes in the pit wall, depressurization can be achieved in a cost-effective way.

- Although the locations of crushed zones were already investigated in Liikavaara Östra, more detailed research is recommended in this topic for a better understanding of the effect of the poor rock quality areas on slope stability and hydrogeology.

- Further research and detailed cost estimation of the external slope support methods are also advised, moreover the development of support application instructions. The assessment of soil or rock like behavior of crushed zones is also recommended.

- To minimize back break in the pit, smooth blasting practices are recommended. With decoupled explosives and carefully designed drill patterns, reduced back break can be achieved. These techniques are advised in all domains. Moreover, for the footwall domains presplit blasting is recommended for best results. Scaling of benches is also proposed to minimize further rock fall.

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List of Appendices

Appendix 1

Bedrock map of Northern Norrbotten.

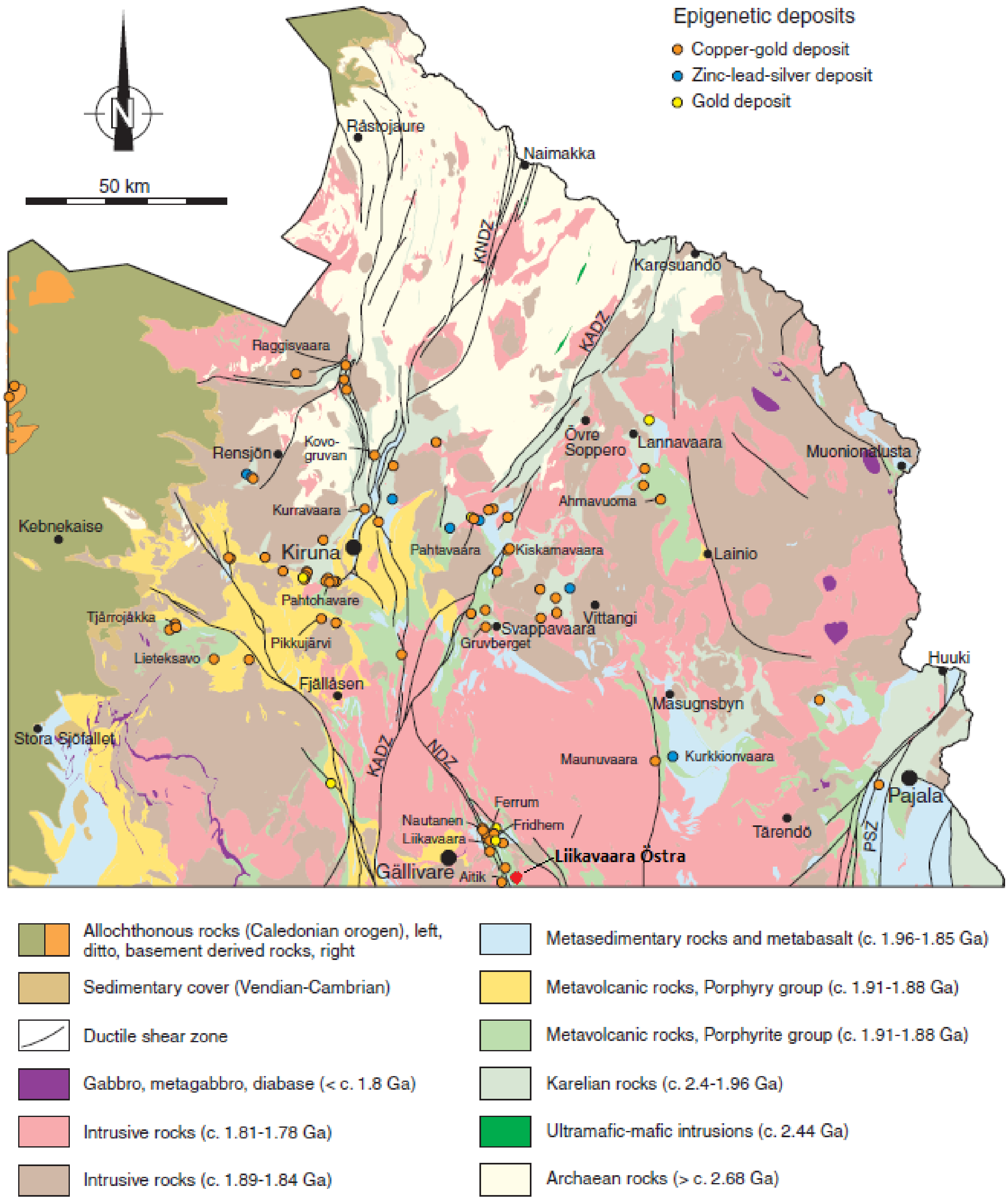


Figure 1-1 Bedrock map of Northern Norrbotten (Bergman, 2001). Referenced to Geographic North.

Appendix 2

Joint orientation data of Aitik and Salmijärvi.

Table 2-1 Joint orientations in Aitik by design sectors (Sjöberg, et al. 2016).

Design Sector	Structure Set	DipDir(°)	ΔDipDir	Dip(°)	ΔDip
Footwall North	JS1(*)	190	11	60	5
	JS2	236	7	84	4
	JS3	56	6	87	3
Footwall Middle	JS1	189	11	58	6
	JS2	238	7	83	3
Footwall South	JS1(*)	191	14	40	7
	JS2	249	7	81	4
	JS3	192	5	82	2
	JS4	73	4	88	2
Hangingwall North	JS1(*)	184	7	46	4
	JS2	69	6	82	3
	JS3	42	7	72	3
	JS4	11	6	71	3
	JS5	341	4	77	2
Hangingwall Middle	JS1(*)	180	8	45	4
	JS2	22	11	75	5
	JS3	67	4	86	2
Hangingwall South	JS1	22	10	82	5
	JS2	257	7	84	4
	JS3	73	6	87	3
	JS4(*)	166	7	44	3
	JS5	334	5	74	2
North End	JS1(*)	183	4	52	2
	JS2	262	6	86	3
	JS3	77	8	82	4

(*) System corresponds to South-dipping foliation.

Table 2-2 Joint orientations in Salmijärvi by design sectors (Perks, 2015).

Design sector	Structure set	Dip direction (°)	Δ Dip direction (°)	Dip (°)	Std. Deviation (°)
CD1.0	JS 1 (*)	175	13	66	6
	JS 2	252	9	84	5
	JS 3	86	8	85	4
	JS 4	327	7	83	4
	JS 5	356	4	2	2
CD2.0	JS 1 (*)	194	8	65	4
	JS 2	270	6	85	3
	JS 3	155	6	61	3
	JS 4	18	8	9	4
LBD2	JS 1	259	8	84	4
	JS 2 (*)	168	9	69	4
	JS 3	284	7	40	3
	JS 4	6	4	76	2
CD3A	JS 1	258	6	87	3
	JS 2	323	9	83	4
	JS 3	150	5	86	2
	JS 4	75	5	87	3
	JS 5	12	7	82	3
SED1	JS 1 (*)	162	8	61	4
	JS 2	270	9	84	4
	JS 3	70	11	85	5
	JS 4	306	6	55	3
	JS 5	356	10	16	5

Appendix 3

Stereonets of Liikavaara borehole orientation data.

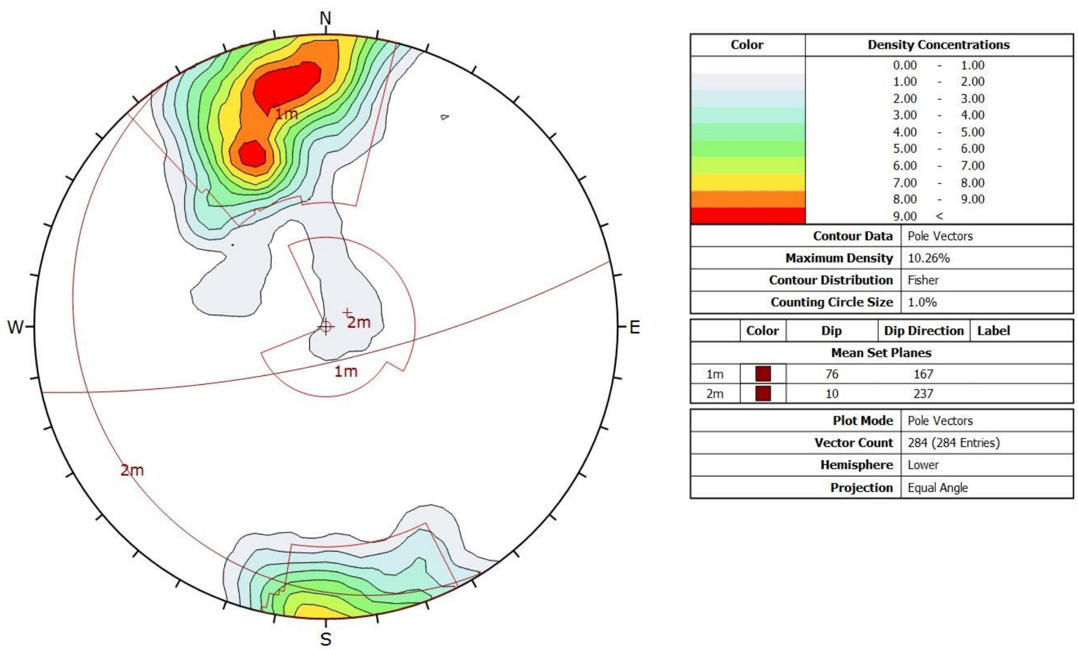


Figure 3-1 Stereonet result of AITIK331 hole with assigned joint sets.

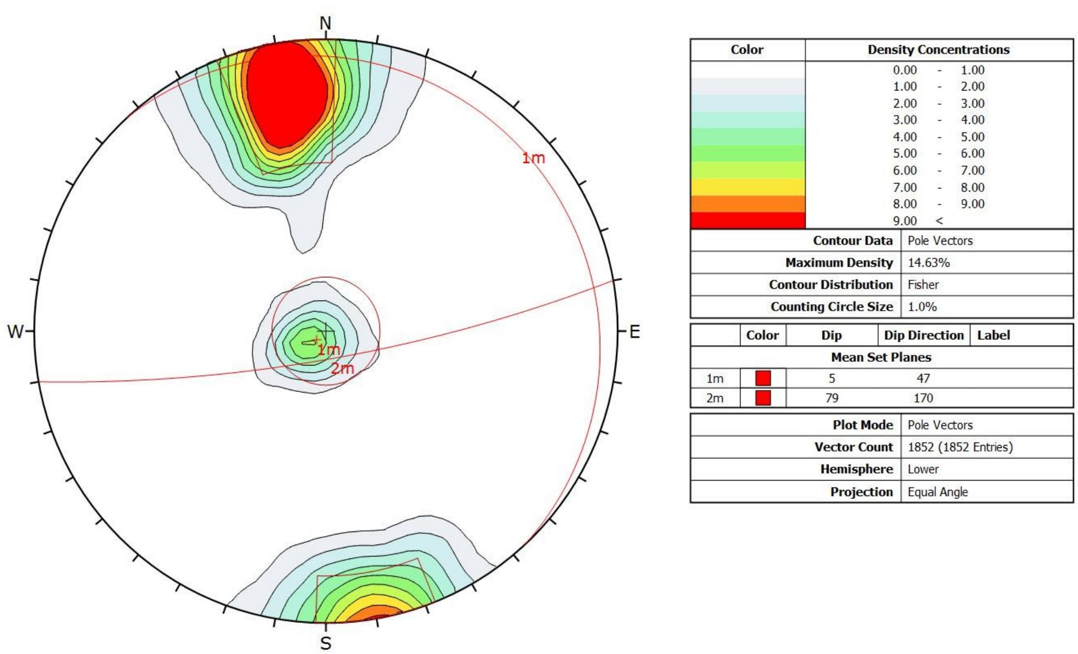
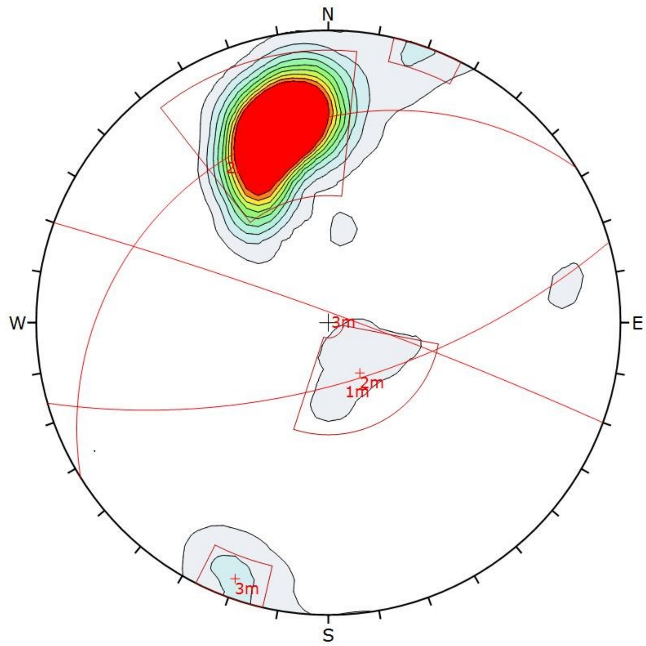


Figure 3-2 Stereonet result of AITIK333 hole with assigned joint sets.

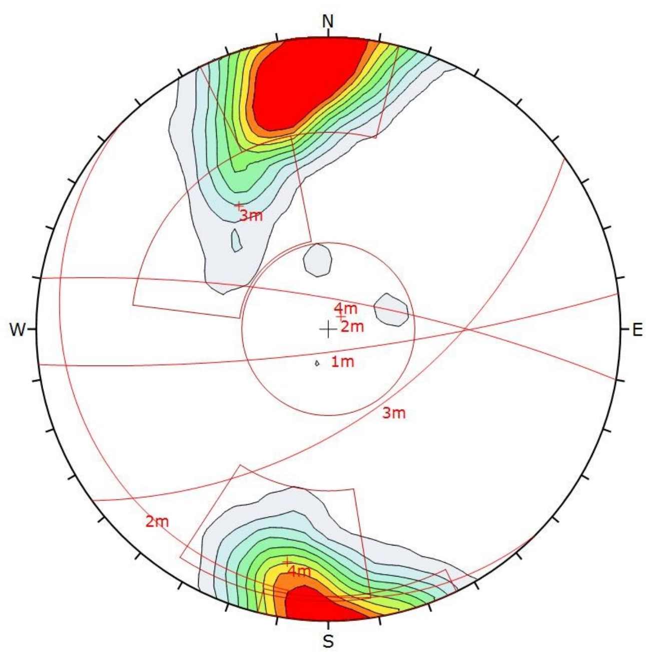


Color	Density Concentrations
	0.00 - 1.00
	1.00 - 2.00
	2.00 - 3.00
	3.00 - 4.00
	4.00 - 5.00
	5.00 - 6.00
	6.00 - 7.00
	7.00 - 8.00
	8.00 - 9.00
	9.00 <
Contour Data	Pole Vectors
Maximum Density	22.05%
Contour Distribution	Fisher
Counting Circle Size	1.0%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m		66	164	
2m		23	328	
3m		86	20	

Plot Mode	Pole Vectors
Vector Count	776 (776 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 3-3 Stereonet result of AITIK363 hole with assigned joint sets.



Color	Density Concentrations
	0.00 - 1.00
	1.00 - 2.00
	2.00 - 3.00
	3.00 - 4.00
	4.00 - 5.00
	5.00 - 6.00
	6.00 - 7.00
	7.00 - 8.00
	8.00 - 9.00
	9.00 <
Contour Data	Pole Vectors
Maximum Density	13.05%
Contour Distribution	Fisher
Counting Circle Size	1.0%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m		81	173	
2m		7	225	
3m		55	144	
4m		78	10	

Plot Mode	Pole Vectors
Vector Count	868 (868 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 3-4 Stereonet result of AITIK365 hole with assigned joint sets.

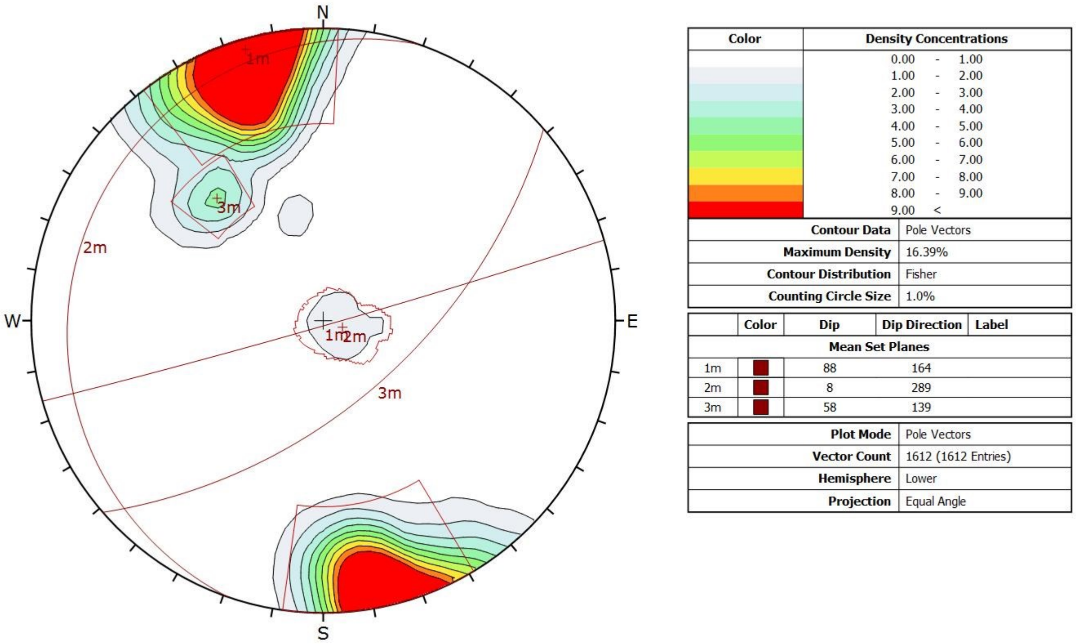


Figure 3-5 Stereonet result of AITIK369 hole with assigned joint sets.

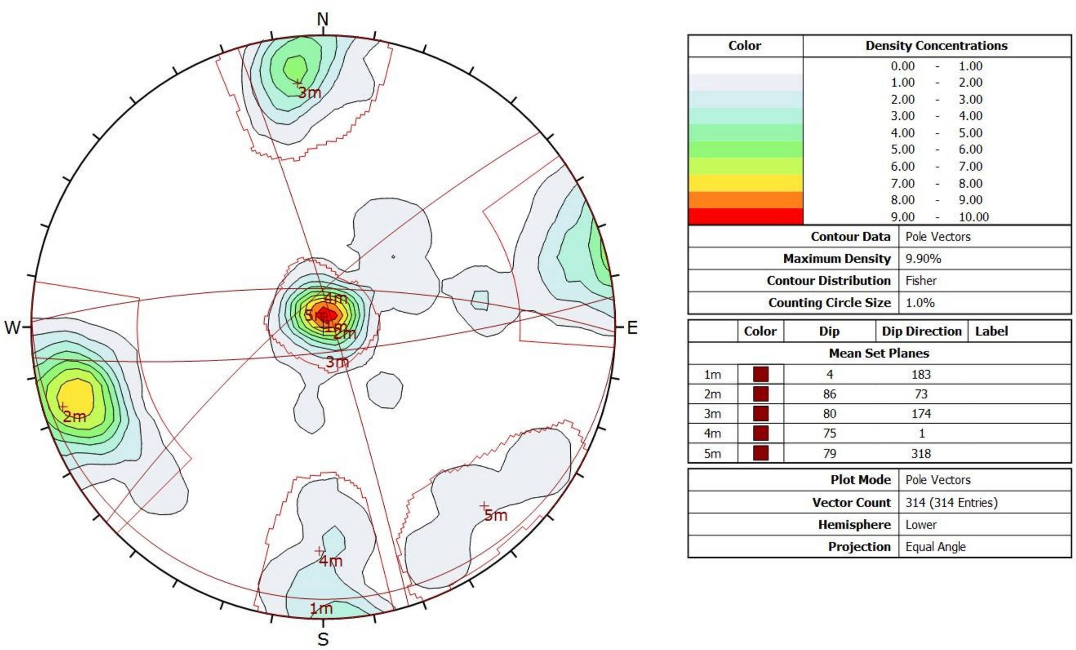


Figure 3-6 Stereonet result of AITIK391 hole with assigned joint sets.

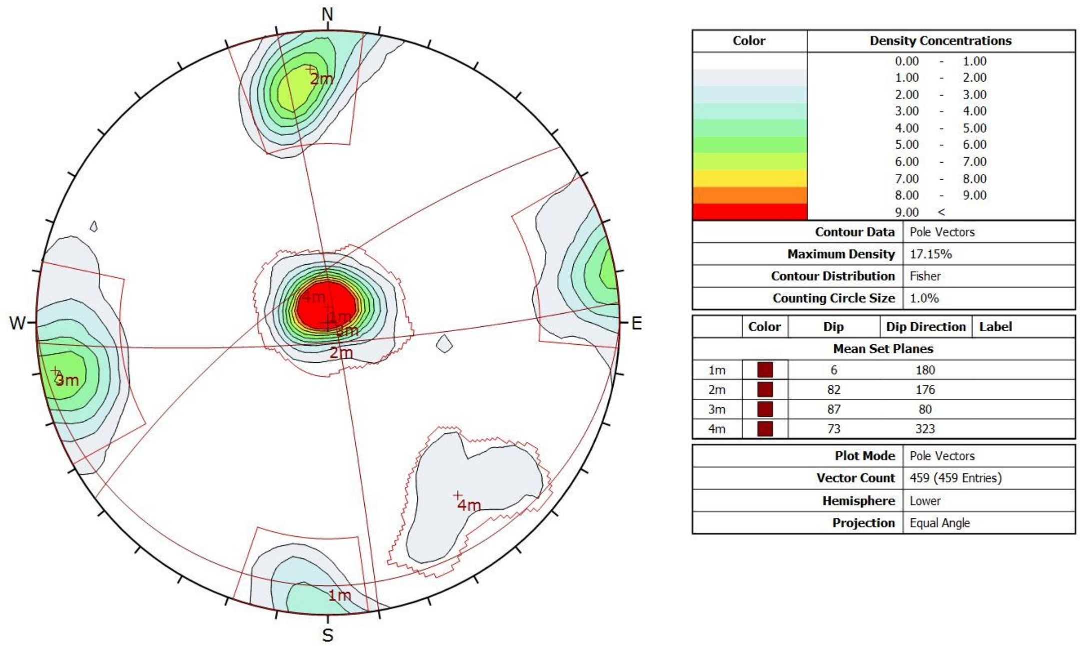


Figure 3-7 Stereonet result of AITIK392 hole with assigned joint sets.

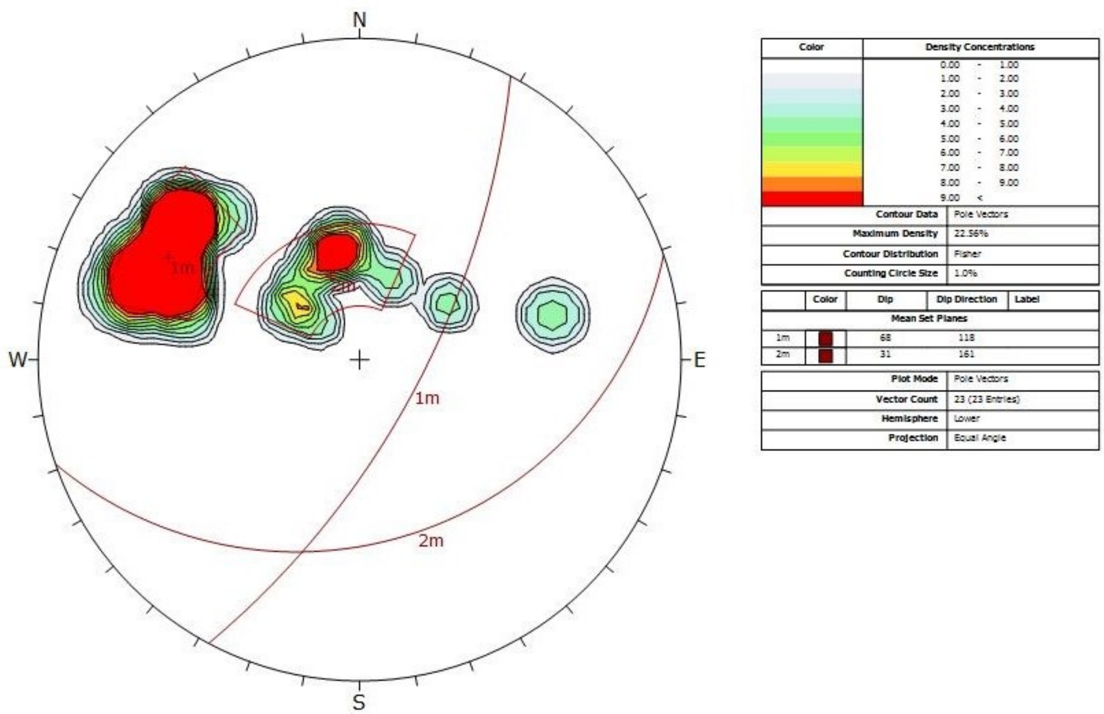


Figure 3-8 Stereonet result of AITIK413 hole with assigned joint sets.

Appendix 4

Kinematic analysis tables.

Table 4-1 Kinematic analysis for temporary domains, wedge failure case.

Kinematic analysis - WEDGE FAILURE Temporary domains						
Domain	Slope Dip Direction [°]	Type of Angle	Slope Angle [°]	Friction Angle [°]	Percentage of Total Poles [%]	BOREHOLE
NE2-lower parts	217	BFA	70	30	11.9	AITIK365
			80	30	17.5	
			90	30	24.8	
		IRA	38	30	1.2	
FW1 - DEEP	146	BFA	70	31	17.8	AITIK369
			80	31	35.4	
			90	31	61.5	
		IRA	48	31	0.2	
FW2 - DEEP	156	BFA	70	30	10.0	AITIK369
			80	30	32.7	
			90	30	67.0	
		IRA	48	30	0.6	

Table 4-2 Kinematic analysis for final domains, wedge failure case.

Kinematic analysis - WEDGE FAILURE Final domains						
Domain	Slope Dip Direction [°]	Type of Angle	Slope Angle [°]	Friction Angle [°]	Percentage of Total Poles [%]	BOREHOLE
WE1	30	BFA	70	30	8.6	AITIK333
			80	30	12.9	
			90	30	19.3	
		IRA	48	30	1.8	
WE2	130	BFA	70	30	16.4	AITIK333
			80	30	22.9	
			90	30	28.2	
		IRA	48	30	5.0	
EE	274	BFA	70	30	11.0	AITIK331
			80	30	14.2	
			90	30	18.4	
		IRA	48	30	5.0	
FW1	146	BFA	70	31	52.8	AITIK413
			80	31	62.3	
			90	31	70.2	
		IRA	48	31	25.8	
FW2	156	BFA	70	30	53.6	AITIK413
			80	30	61.1	
			90	30	62.7	
		IRA	48	30	40.1	
NE1	185	BFA	70	30	26.8	AITIK363
			80	30	44.6	
			90	30	56.7	
		IRA	38	30	1.2	
NE2	217	BFA	70	30	16.8	AITIK363
			80	30	23.1	
			90	30	29.5	
		IRA	38	30	2.3	
HW1	319	BFA	70	35	5.3	AITIK392
			80	35	9.1	
			90	35	13.0	
		IRA	56	35	1.1	
HW2	9	BFA	70	35	8.3	AITIK391
			80	35	12.9	
			90	35	19.1	
		IRA	56	35	3.2	

Table 4-3 Kinematic analysis for final domains, flexural toppling failure case.

Kinematic analysis - FLEXURAL TOPPLING FAILURE Final domains						
Domain	Slope Dip Direction [°]	Type of Angle	Slope Angle [°]	Friction Angle [°]	Percentage of Total Poles [%]	BOREHOLE
WE1	30	BFA	70	30	3.9	AITIK333
			80	30	4.1	
			90	30	4.5	
		IRA	48	30	2.5	
WE2	130	BFA	70	30	1.2	AITIK333
			80	30	1.4	
			90	30	1.6	
		IRA	48	30	1.1	
EE	274	BFA	70	30	0.7	AITIK331
			80	30	2.5	
			90	30	2.8	
		IRA	48	30	0.0	
FW1	146	BFA	70	31	0.0	AITIK413
			80	31	0.0	
			90	31	0.0	
		IRA	48	31	0.0	
FW2	156	BFA	70	30	0.0	AITIK413
			80	30	0.0	
			90	30	0.0	
		IRA	48	30	0.0	
NE1	185	BFA	70	30	4.1	AITIK363
			80	30	4.3	
			90	30	5.4	
		IRA	38	30	1.9	
NE2	217	BFA	70	30	3.5	AITIK363
			80	30	3.9	
			90	30	4.0	
		IRA	38	30	1.8	
HW1	319	BFA	70	35	1.5	AITIK392
			80	35	1.7	
			90	35	2.2	
		IRA	56	35	1.5	
HW2	9	BFA	70	35	9.2	AITIK391
			80	35	10.2	
			90	35	11.5	
		IRA	56	35	8.9	

Table 4-4 Kinematic analysis for temporary domains, flexural toppling failure case.

Kinematic analysis – FLEXURAL TOPPLING FAILURE Temporary domains						
Domain	Slope Dip Direction [°]	Type of Angle	Slope Angle [°]	Friction Angle [°]	Percentage of Total Poles [%]	BOREHOLE
NE2 lower parts	217	BFA	70	30	4.7	AITIK365
			80	30	4.7	
			90	30	4.7	
		IRA	38	30	1.4	
FW1 – DEEP	146	BFA	70	31	13.9	AITIK369
			80	31	13.9	
			90	31	13.9	
		IRA	48	31	13.1	
FW2 – DEEP	156	BFA	70	30	18.4	AITIK369
			80	30	18.4	
			90	30	18.4	
		IRA	48	30	17.2	

Table 4-5 Kinematic analysis for final domains, direct toppling failure case.

Kinematic analysis – DIRECT TOPPLING FAILURE Final domains						
Domain	Slope Dip Direction [°]	Type of Angle	Slope Angle [°]	Friction Angle [°]	Percentage of Total Poles [%]	BOREHOLE
WE1	30	BFA	70	30	3.7	AITIK333
			80	30	3.9	
			90	30	4.4	
		IRA	48	30	3.1	
WE2	130	BFA	70	30	0.7	AITIK333
			80	30	1.0	
			90	30	1.8	
		IRA	48	30	0.6	
EE	274	BFA	70	30	30.3	AITIK331
			80	30	33.6	
			90	30	37.5	
		IRA	48	30	16.3	
FW1	146	BFA	70	31	0.0	AITIK413
			80	31	0.0	
			90	31	0.0	
		IRA	48	31	0.0	
FW2	156	BFA	70	30	0.0	AITIK413
			80	30	0.0	
			90	30	0.0	
		IRA	48	30	0.0	
NE1	185	BFA	70	30	0.5	AITIK363
			80	30	0.8	
			90	30	1.2	
		IRA	38	30	0.1	
NE2	217	BFA	70	30	0.2	AITIK363
			80	30	0.4	
			90	30	1.0	
		IRA	38	30	0.1	
HW1	319	BFA	70	35	3.6	AITIK392
			80	35	5.2	
			90	35	10.3	
		IRA	56	35	3.3	
HW2	9	BFA	70	35	3.6	AITIK391
			80	35	5.7	
			90	35	10.4	
		IRA	56	35	2.5	

Table 4-6 Kinematic analysis for temporary domains, direct toppling failure case.

Kinematic analysis - DIRECT TOPPLING FAILURE Temporary domains						
Domain	Slope Dip Direction [°]	Type of Angle	Slope Angle [°]	Friction Angle [°]	Percentage of Total Poles [%]	BOREHOLE
NE2 lower parts	217	BFA	70	30	1.1	AITIK365
			80	30	1.4	
			90	30	1.7	
		IRA	38	30	1.0	
FW1 - DEEP	146	BFA	70	31	0.8	AITIK369
			80	31	1.1	
			90	31	1.2	
		IRA	48	31	0.5	
FW2 - DEEP	156	BFA	70	30	0.8	AITIK369
			80	30	1.0	
			90	30	1.2	
		IRA	48	30	0.5	

Appendix 5

Probabilistic analysis tables.

Table 5-1 Results of probabilistic analysis, undrained Barton-Bandis scenario.

Barton- Bandis			Undrained				
Domain	**Drill hole**	**Bench height [m]**	**Analyzed sets of hole**	**Average slope length [m]**	**Effective bench width [m]**	**Wedge failure BFA @ 15% PoF**	**Planar failure BFA @ 15% PoF**
WE1	AITIK333	30	1-2	137	11	85	-
WE2	AITIK333	30	1-2	157	11	85	-
FW1	AITIK413	30	1-2	157	11	32	27
FW2	AITIK413	30	1-2	111	11	29	27
NE1	AITIK363	30	1-2	124	11	85	60
NE2	AITIK363	30	1-2	167	11	85	-
EE	AITIK331	30	1-2	93	11	22	-
HW1	AITIK392	30	1-4	315	11	85	72
	AITIK392	30	2-4	315	11	70	
	AITIK392	30	3-4	315	11	64	
HW2	AITIK391	30	1-4	176	11	85	66
	AITIK391	30	2-4	176	11	66	
	AITIK391	30	2-5	176	11	63	
	AITIK391	30	3-4	176	11	85	
	AITIK391	30	4-5	176	11	65	
FW1 & FW2 DEEP	AITIK369	30	1-2	157 / 111	11	65	55
NE2 lower parts	AITIK365	30	1-2	167	11	38	-
	AITIK365	30	1-4	167	11	85	-

Table 5-2 Results of probabilistic analysis, drained Mohr-Coulomb scenario.

Mohr-Coulomb			Drained				
Domain	**Drill hole**	**Bench height [m]**	**Analyzed sets of hole**	**Average slope length [m]**	**Effective bench width [m]**	**Wedge failure BFA @ 15% PoF**	**Planar failure BFA @ 15% PoF**
WE1	AITIK333	30	1-2	137	11	85	-
WE2	AITIK333	30	1-2	157	11	85	-
FW1	AITIK413	30	1-2	157	11	38	27
FW2	AITIK413	30	1-2	111	11	36	27
NE1	AITIK363	30	1-2	124	11	85	58
NE2	AITIK363	30	1-2	167	11	85	-
EE	AITIK331	30	1-2	93	11	85	-
HW1	AITIK392	30	1-4	315	11	85	63
	AITIK392	30	2-4	315	11	71	
	AITIK392	30	3-4	315	11	64	
HW2	AITIK391	30	1-4	176	11	85	68
	AITIK391	30	2-4	176	11	66	
	AITIK391	30	2-5	176	11	62	
	AITIK391	30	3-4	176	11	85	
	AITIK391	30	4-5	176	11	67	
FW1 & FW2 DEEP	AITIK369	30	1-2	157 / 111	11	65	55
NE2 lower parts	AITIK365	30	1-2	167	11	85	-
	AITIK365	30	1-4	167	11	85	-

Table 5-3 Results of probabilistic analysis, undrained Mohr-Coulomb scenario.

Mohr-Coulomb			Undrained				
Domain	**Drill hole**	**Bench height [m]**	**Analyzed sets of hole**	**Average slope length [m]**	**Effective bench width [m]**	**Wedge failure BFA @ 15% PoF**	**Planar failure BFA @ 15% PoF**
WE1	AITIK333	30	1-2	137	11	85	-
WE2	AITIK333	30	1-2	157	11	85	-
FW1	AITIK413	30	1-2	157	11	31	27
FW2	AITIK413	30	1-2	111	11	28	27
NE1	AITIK363	30	1-2	124	11	85	58
NE2	AITIK363	30	1-2	167	11	85	-
EE	AITIK331	30	1-2	93	11	25	-
HW1	AITIK392	30	1-4	315	11	56	63
	AITIK392	30	2-4	315	11	61	
	AITIK392	30	3-4	315	11	64	
HW2	AITIK391	30	1-4	176	11	85	68
	AITIK391	30	2-4	176	11	66	
	AITIK391	30	2-5	176	11	62	
	AITIK391	30	3-4	176	11	85	
	AITIK391	30	4-5	176	11	65	
FW1 & FW2 DEEP	AITIK369	30	1-2	157 / 111	11	65	55
NE2 lower parts	AITIK365	30	1-2	167	11	28	-
	AITIK365	30	1-4	167	11	85	-

Appendix 6

Bench design criteria of FW1 and FW2 domains.

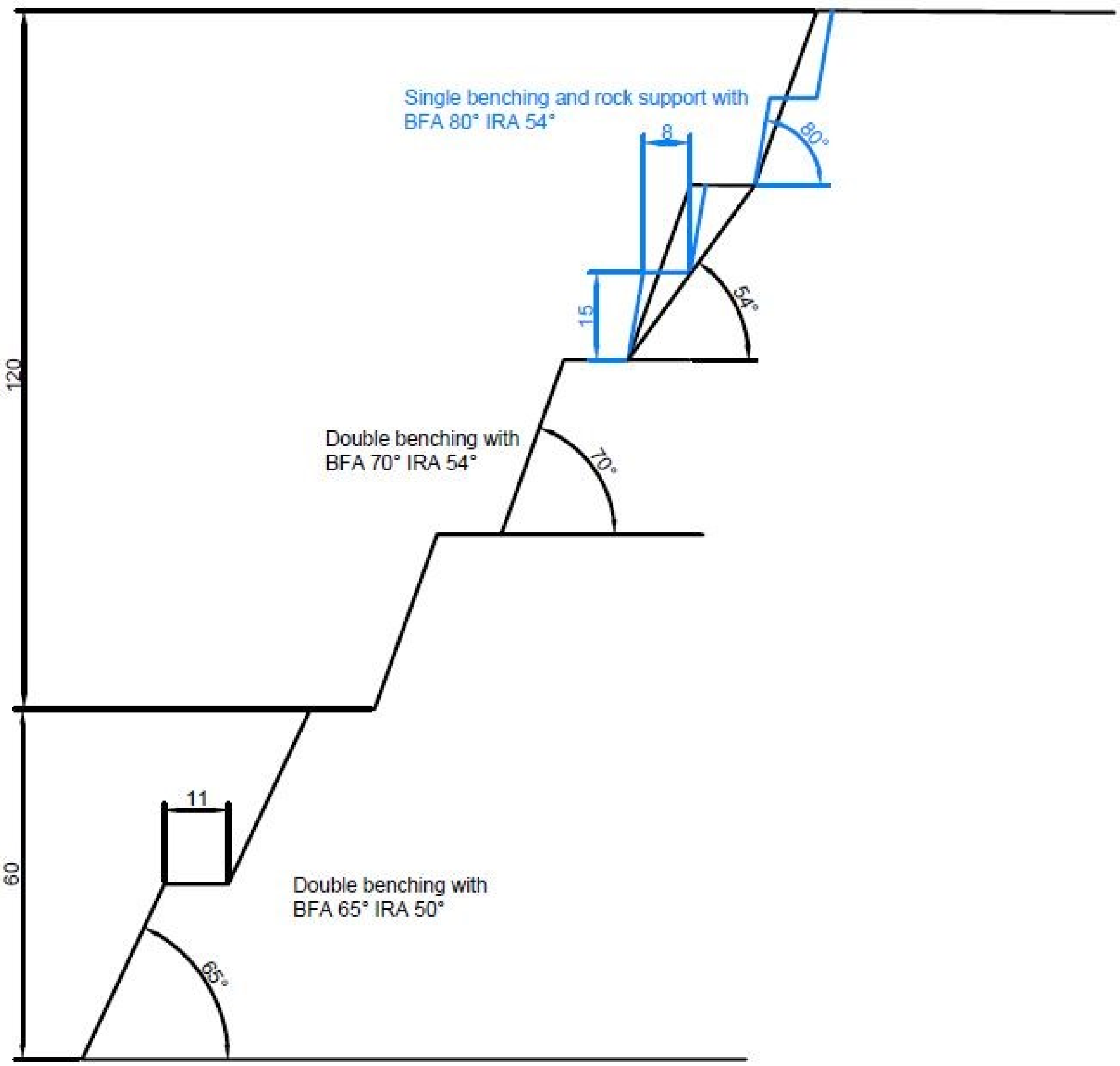


Figure 6-1 Bench design criteria of FW1 and FW2 domains.

Appendix 7

Overall slope stability design sections in *SLIDE 7.0.*

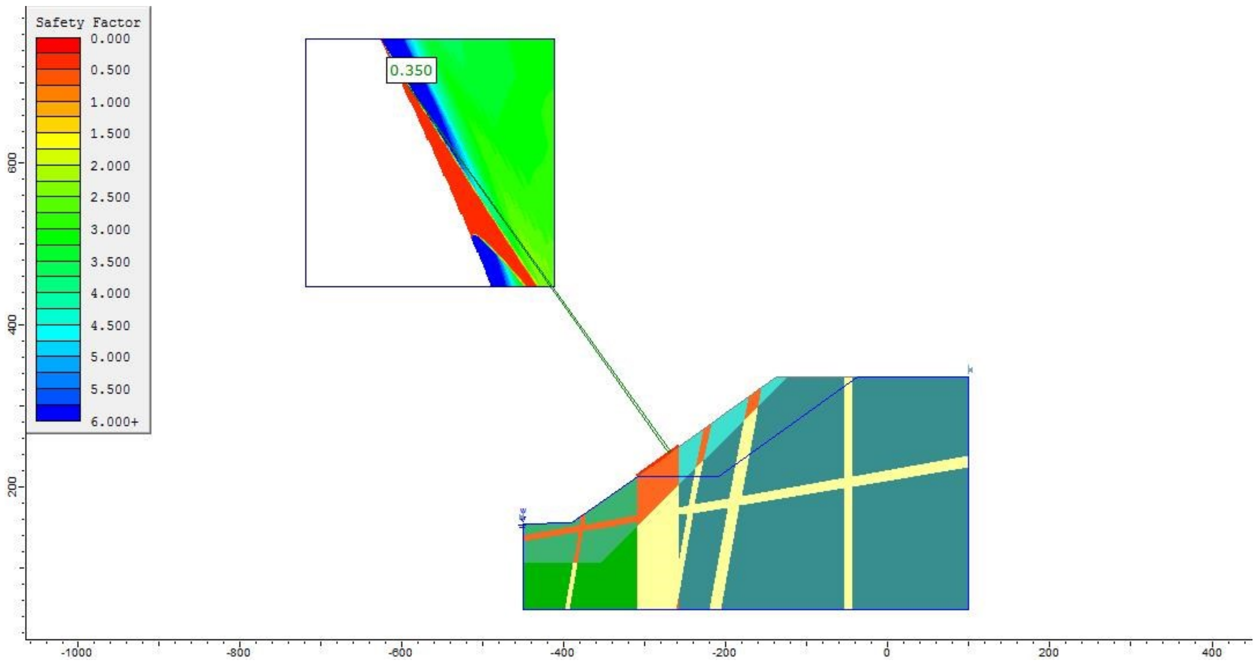


Figure 7-1 Overall slope stability check in the NE design section with partially depressurized slope and OSA 36°.

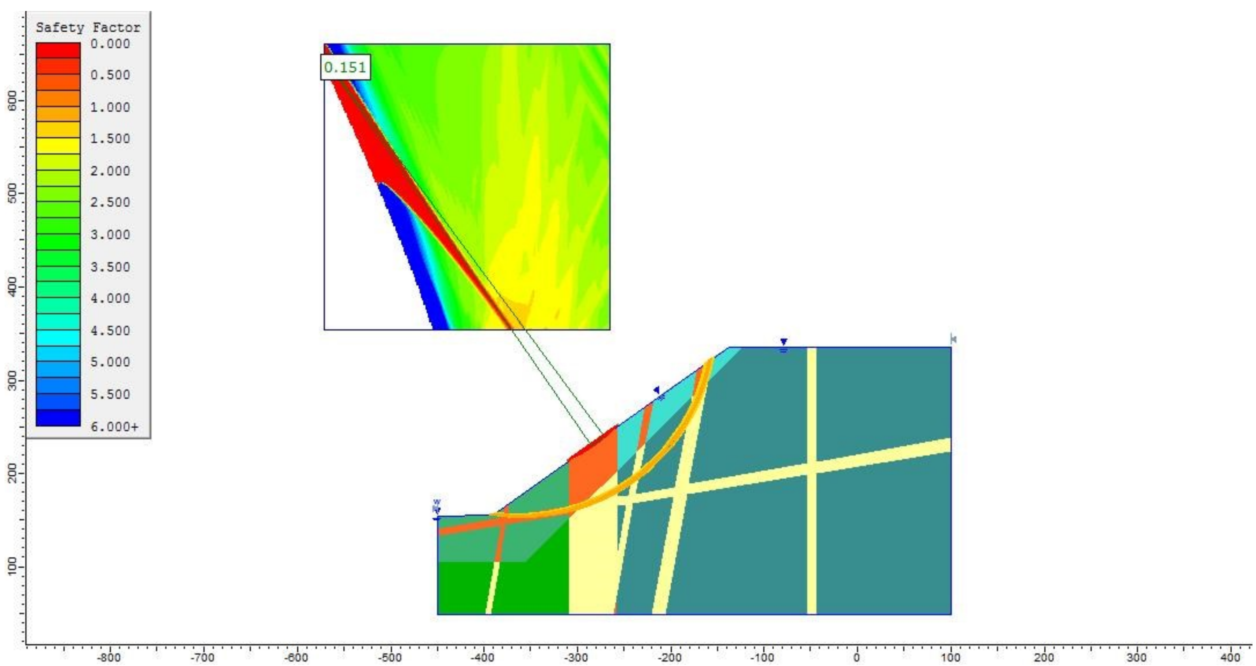


Figure 7-2 Overall slope stability check in the NE design section with undrained slope and OSA 36°.

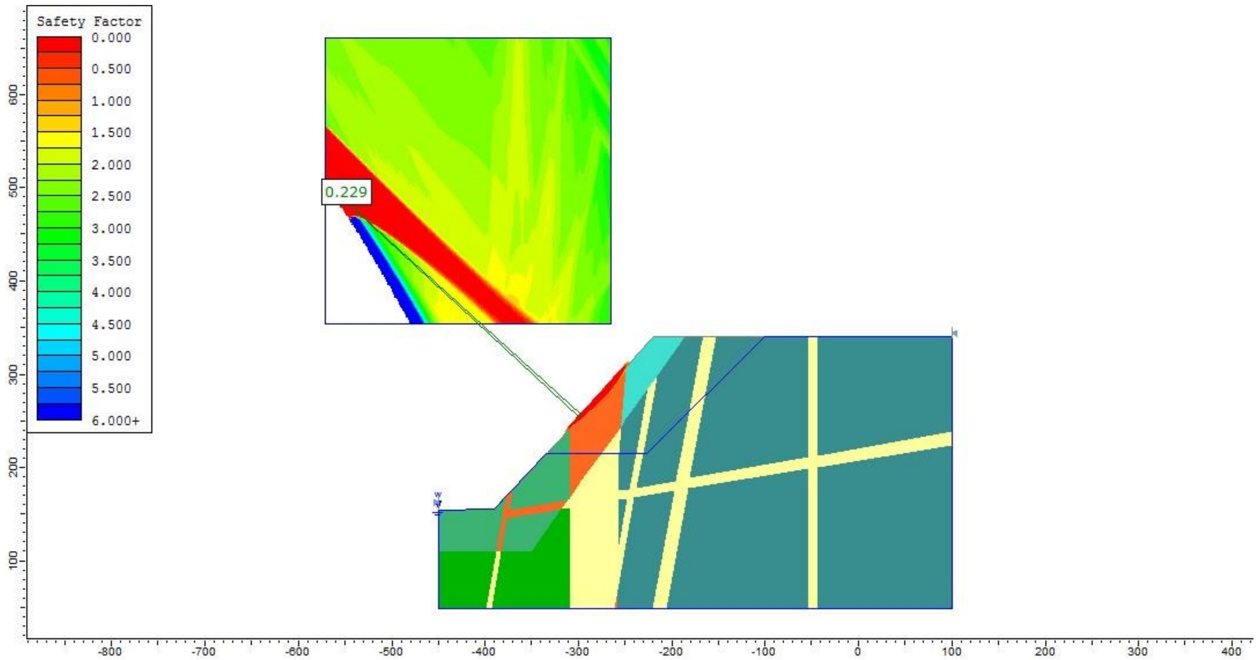


Figure 7-3 Overall slope stability check in the NE design section with partially depressurized slope and OSA 47°.

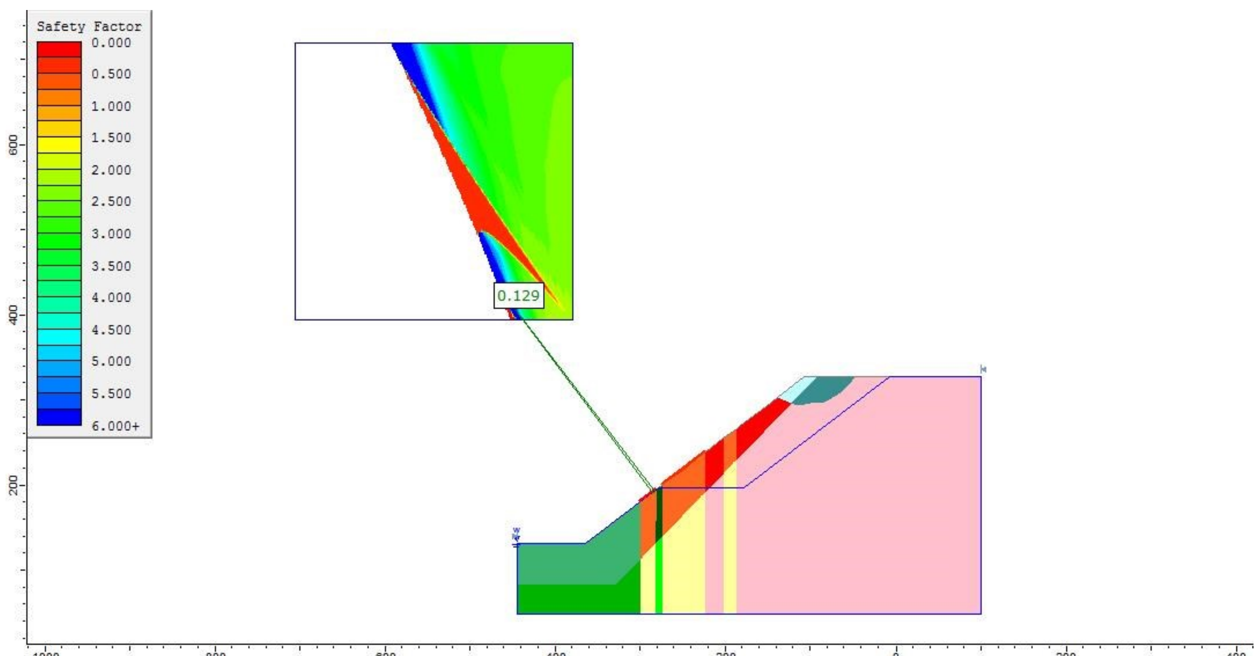


Figure 7-4 Overall slope stability check in the FW design section with partially depressurized slope and OSA 37°.

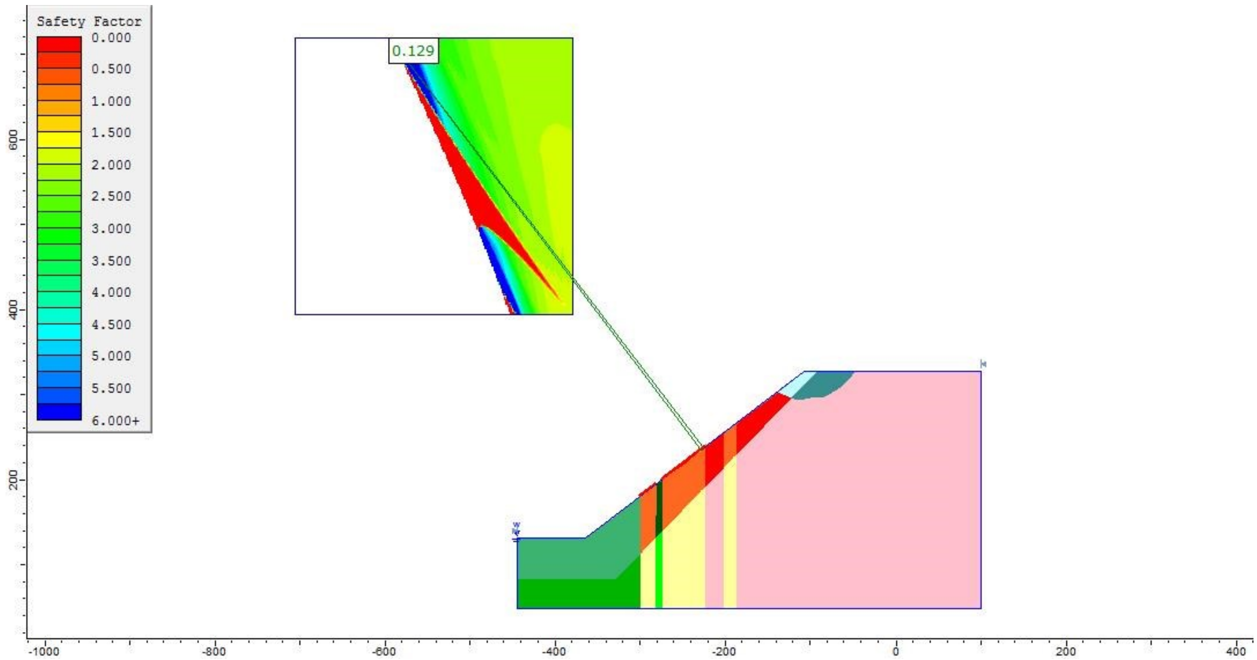


Figure 7-5 Overall slope stability check in the FW design section with undrained slope and OSA 37°.

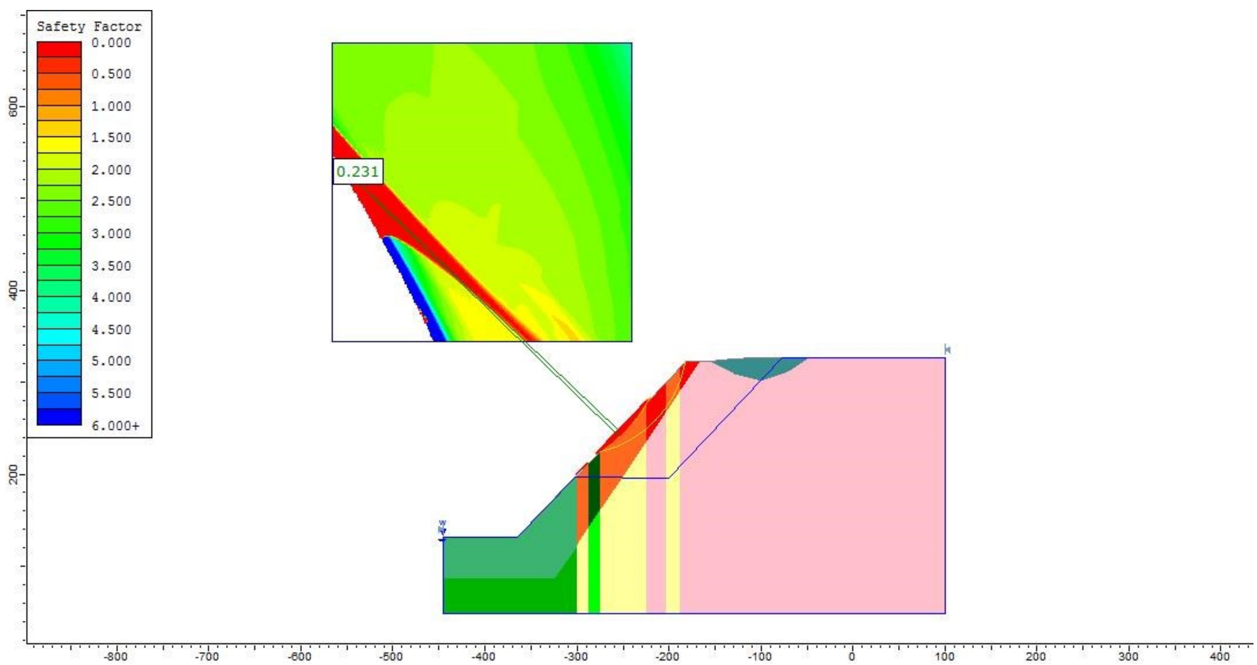


Figure 7-6 Overall slope stability check in the FW design section with partially depressurized slope and OSA 46°.

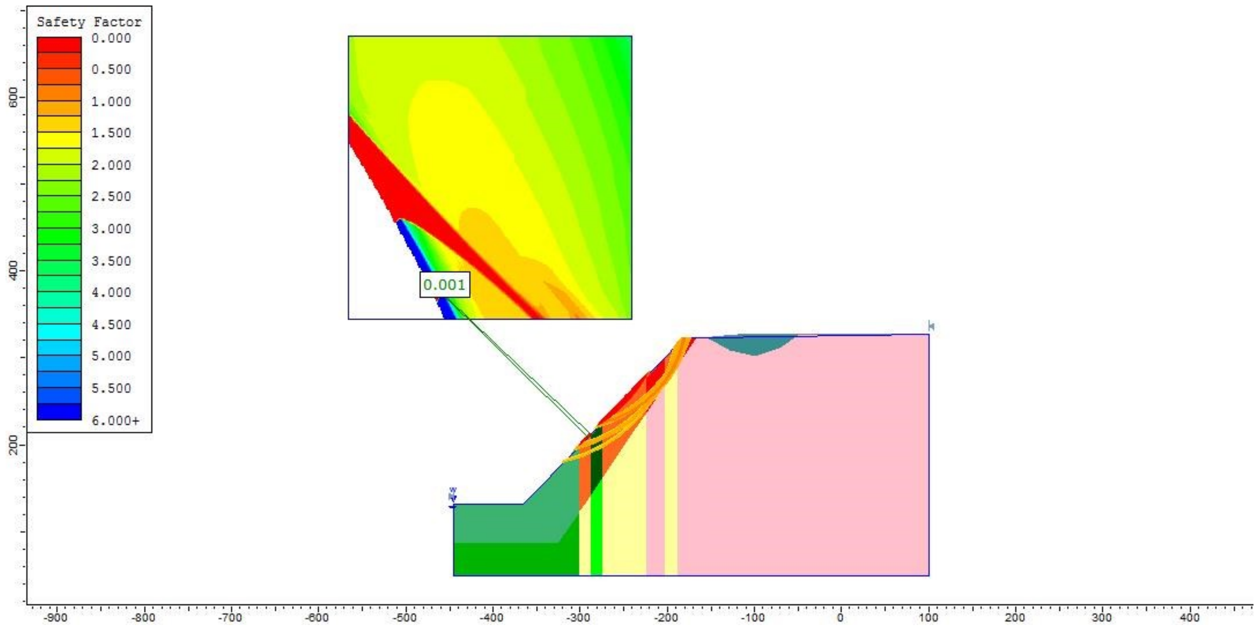


Figure 7-7 Overall slope stability check in the FW design section with undrained slope and OSA 46°.

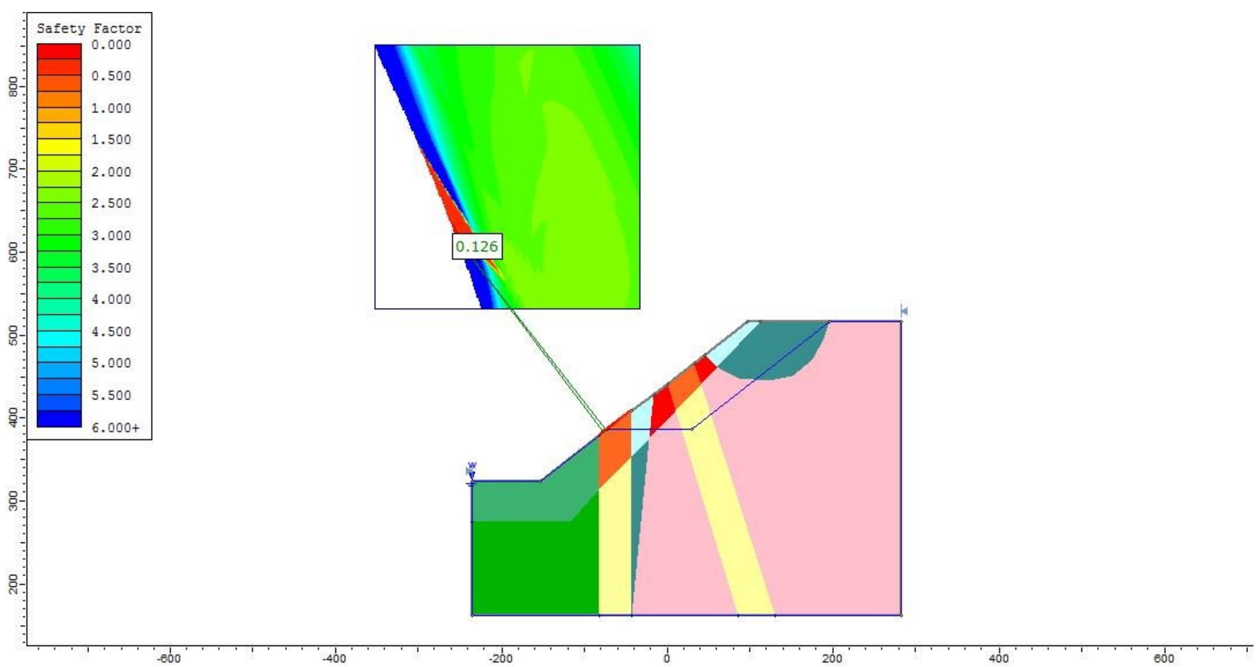


Figure 7-8 Overall slope stability check in the Y4790 design section with partially depressurized slope and OSA 38°.

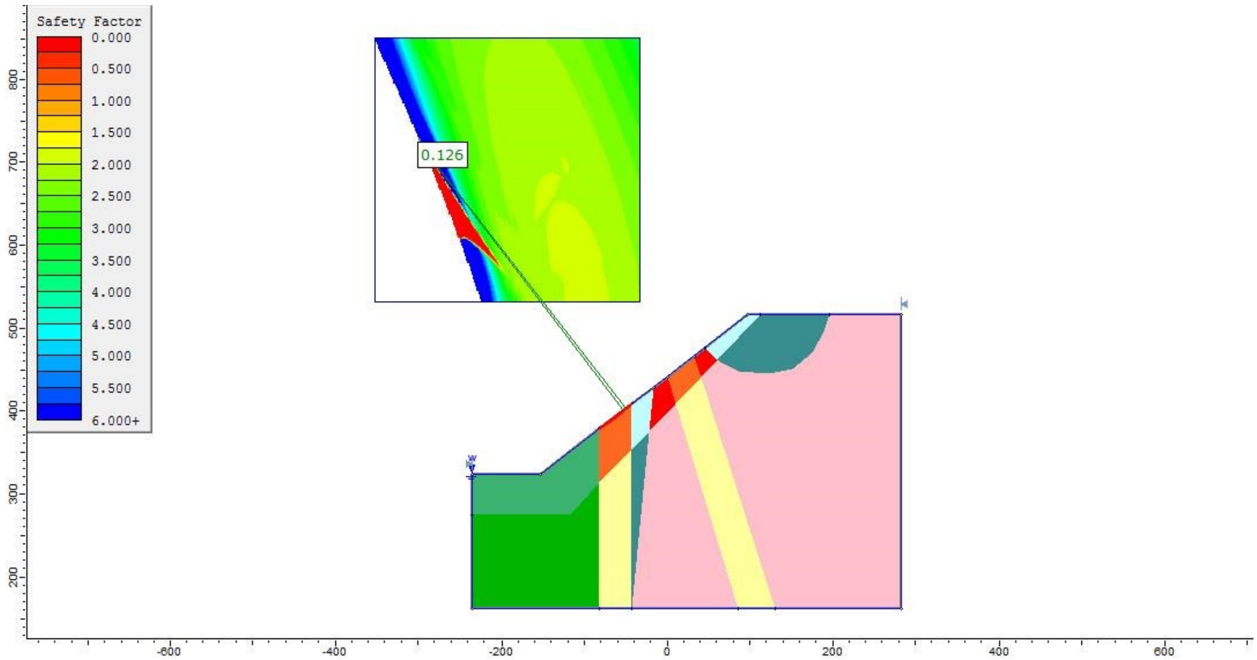


Figure 7-9 Overall slope stability check in the Y4790 design section with undrained slope and OSA 38°.

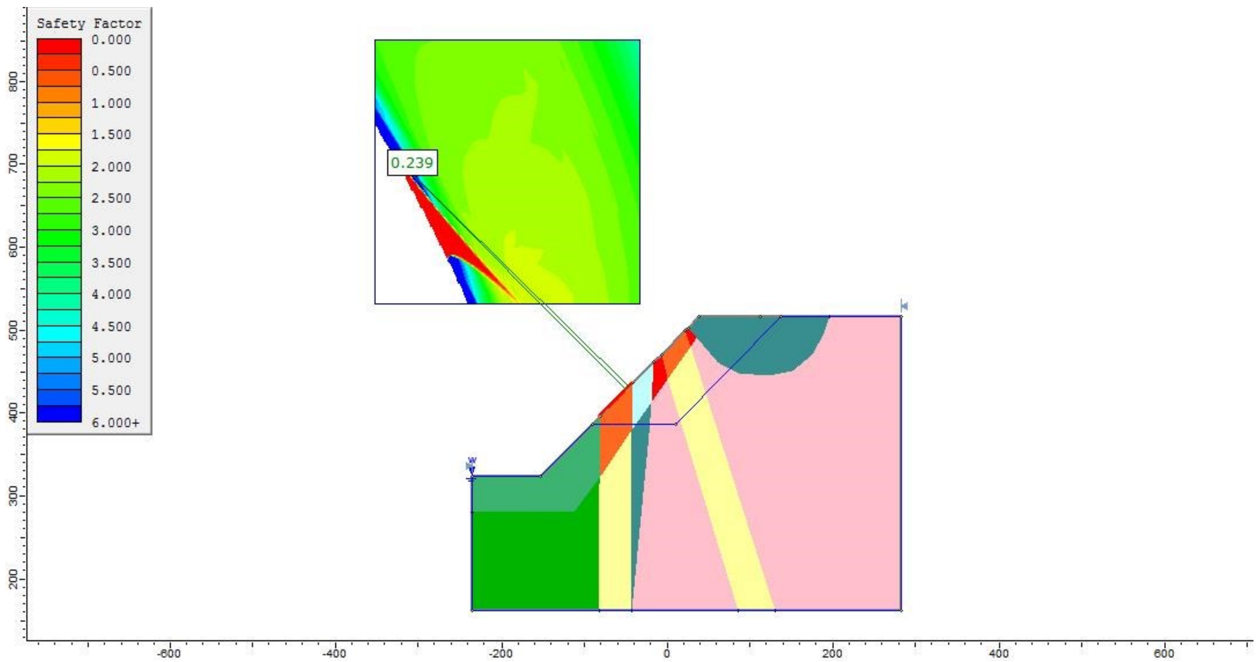


Figure 7-10 Overall slope stability check in the Y4790 design section with partially depressurized slope and OSA 45°.

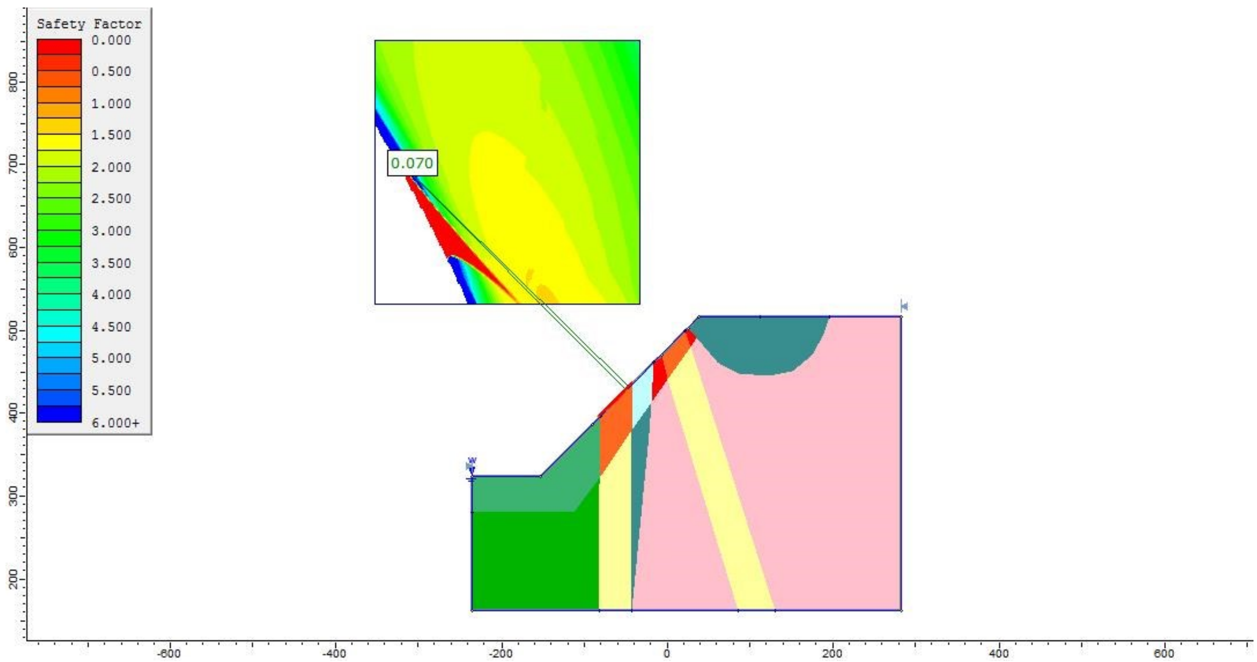
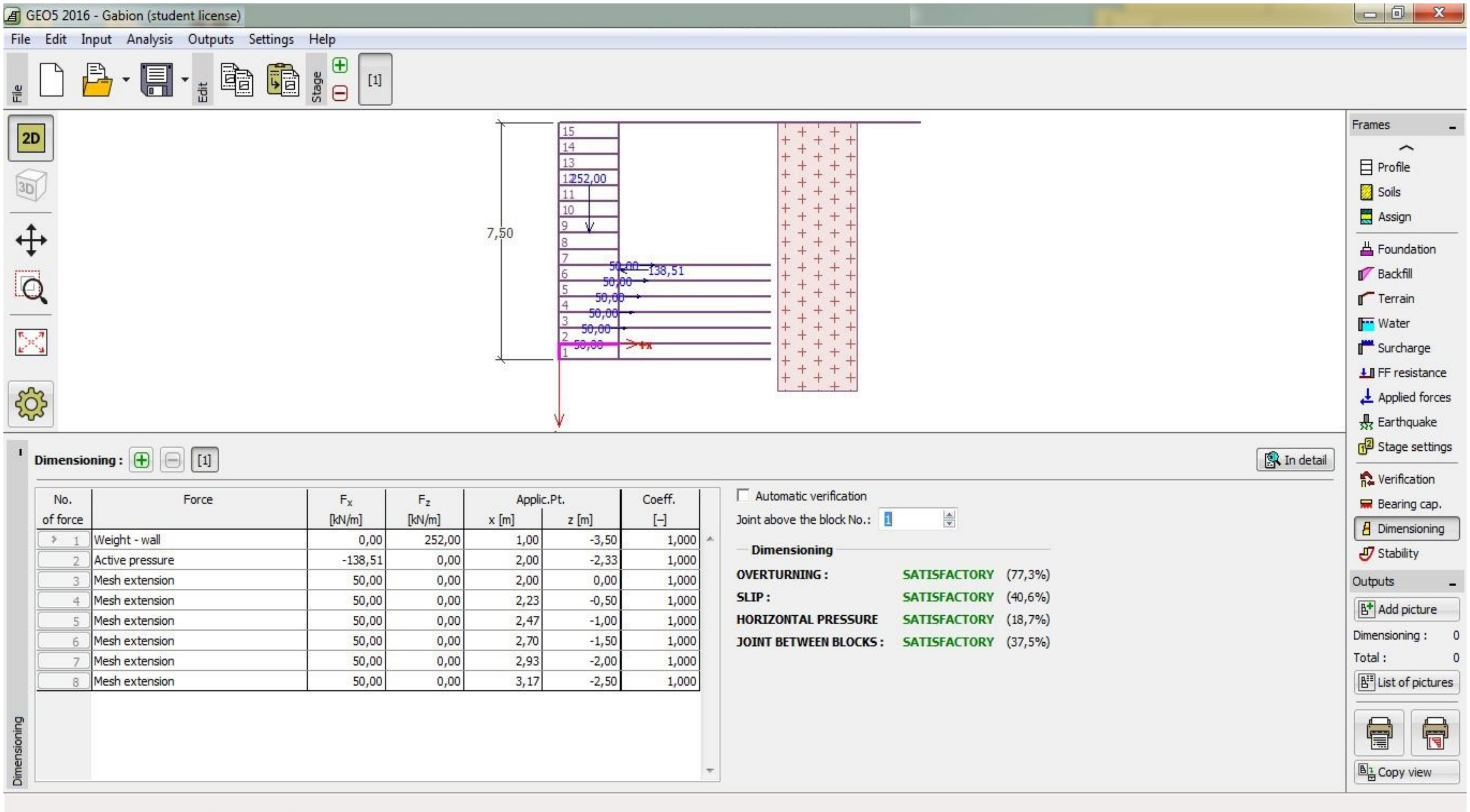


Figure 7-11 Overall slope stability check in the Y4790 design section with undrained slope and OSA 45°.

Appendix 8

External support design in *GEO5*.



No. of force	Force	F_x [kN/m]	F_z [kN/m]	Applic.Pt. x [m]	Applic.Pt. z [m]	Coeff. [–]
1	Weight - wall	0,00	252,00	1,00	-3,50	1,000
2	Active pressure	-138,51	0,00	2,00	-2,33	1,000
3	Mesh extension	50,00	0,00	2,00	0,00	1,000
4	Mesh extension	50,00	0,00	2,23	-0,50	1,000
5	Mesh extension	50,00	0,00	2,47	-1,00	1,000
6	Mesh extension	50,00	0,00	2,70	-1,50	1,000
7	Mesh extension	50,00	0,00	2,93	-2,00	1,000
8	Mesh extension	50,00	0,00	3,17	-2,50	1,000

Figure 8-1 Stability check of single gabion wall with GEO5.

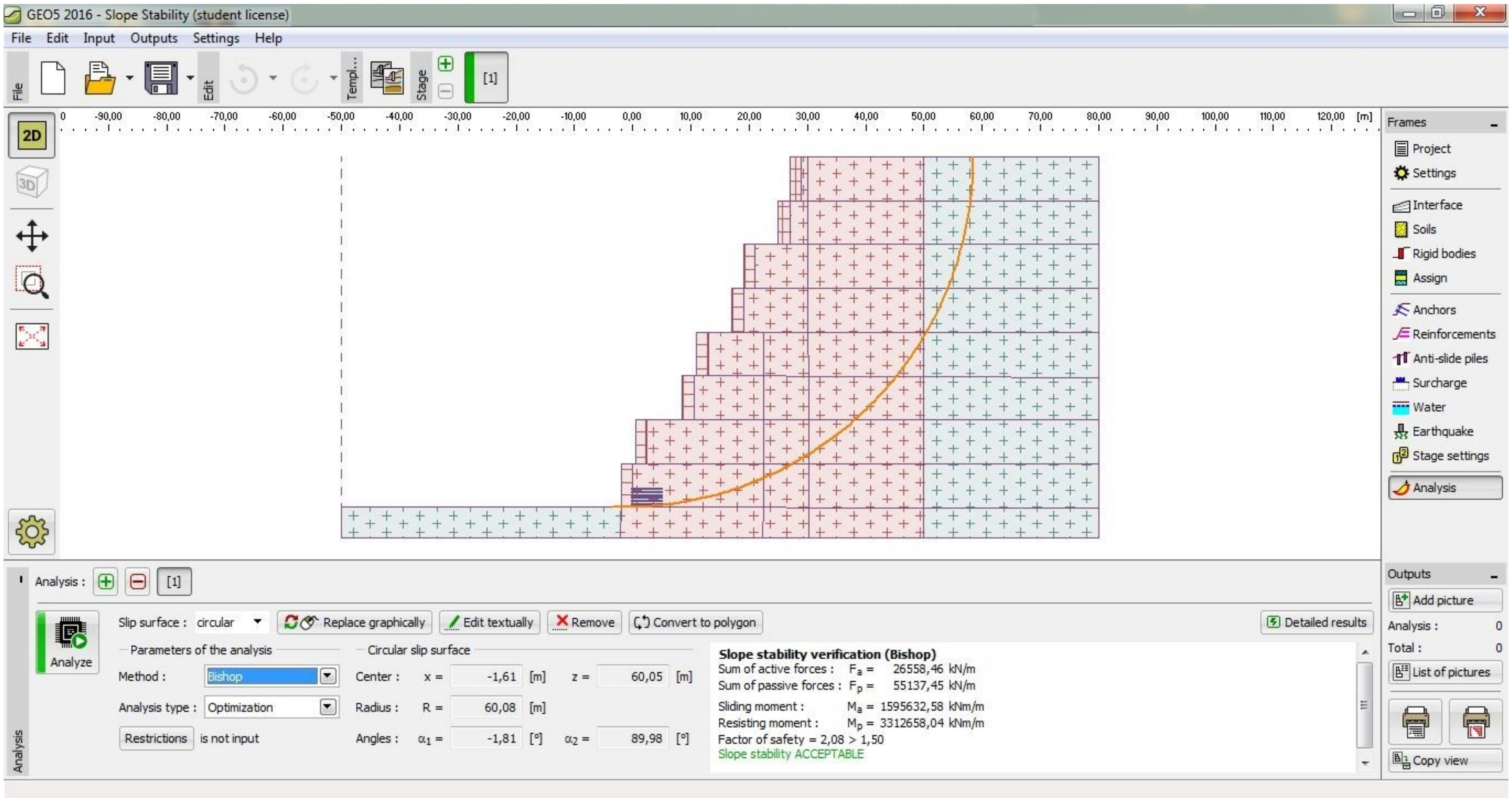


Figure 8-2 Stability check of 60 m high gabion wall with *GEO5*. Factor of Safety with Bishop method is 2.09.

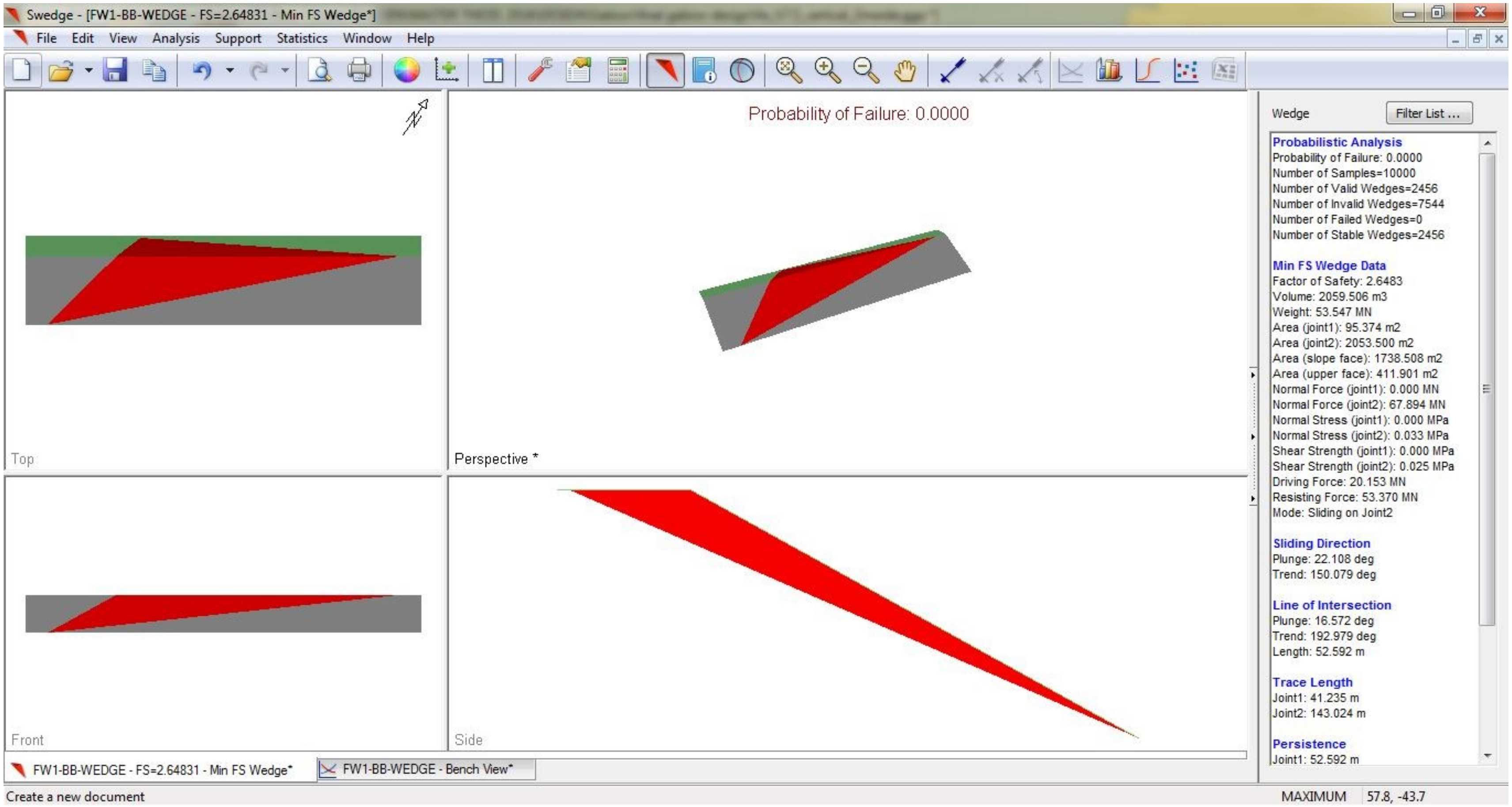


Figure 8-3 Stability check for shotcrete covered slope in *SWEDGE 6.0*. Shotcrete thickness is 0.1 m, tensile strength is 1 MPa. Minimum Factor of Safety is 2.65.

Appendix 9

Cost calculation tables of the external slope support methods.

Table 9-1 Calculation table of reference case and unsupported scenario.

Reference case, Stable @ BFA 80°		
BFA	80	°
IRA	55	°
Bench height	15	m
Bench width	8	m
Excavated area (measured from vertical)	1283	m^2
FW1 bench length	157	m
FW2 bench length	111	m
Waste rock density	2.65	t/m^3
Cost of waste rock mining	25	SEK/t
Total mined waste volume	343 844	m^3
Total mined waste weight	911 187	t
Cost of waste mining	**22 779 665**	**SEK**
Stable @ BFA 36°		
FW upper benches are left in BFAs which is stable without support		
BFA	36	°
IRA	28	°
Bench height	15	m
Bench width	20,3	m
Excavated area	3453	m^2
FW1 bench length	157	m
FW2 bench length	111	m
Waste rock density	2,65	$t/\ m^3$
Cost of waste rock mining	25	SEK/t
Total mined waste volume	925 404	m^3
Total mined waste weigth	2 452 321	t
Total cost of waste mining @ BFA 36°	**61 308 015**	**SEK**
Grand Total (stable @ 36° –reference case)	**38 528 350**	**SEK**
Cost of moving E10 (approx.)	**100 000 000**	**SEK**
Total cost (mining + road relocation)	**161 308 015**	**SEK**
Grand Total (stable @ 36° –reference case)	**138 528 350**	**SEK**

Table 9-2 Calculation table of gabion retaining wall method.

Gabion retaining wall		
BFA	80	°
IRA	54.6	°
Bench height	15	m
Bench width	8	m
Excavated area	1283	m^2
FW1 bench length	157	m
FW2 bench length	111	m
Waste rock density	2.65	t/ m^3
Cost of waste rock mining	25	SEK/t
Total mined waste volume	343 844	m^3
Total mined waste weight	911 187	t
Cost of waste mining	**22 779 665**	**SEK**
Gabion cage area (15 m bench height)	30	m^2
Total gabion cage area for 4 benches	120	m^2
Gabion built in FW1	18 840	m^3
Gabion built in FW2	13 320	m^3
Gabion construction cost / m^3	**700**	**SEK/ m^3**
Cost of gabion in FW1	**13 188 000**	**SEK**
Cost of gabion in FW2	**9 324 000**	**SEK**
Total cost (gabion + waste mining)	**45 291 665**	**SEK**
Grand Total (total cost- reference case)	**22 512 000**	**SEK**
Cost per meter of supported bench	**21 000**	**SEK/m**

Table 9-3 Calculation table of shotcrete, mesh and bolting support method 1/2.

Shotcrete 10cm + Mesh + Bolts		
BFA	80.0	°
IRA	54.6	°
Bench height	15.0	m
Bench width	8.0	m
Number of benches	4.0	
Excavated area	1 283.0	m^2
FW1 bench length	157.0	m
FW2 bench length	111.0	m
Waste rock density	2.7	t/m^3
Cost of waste rock mining	25.0	SEK/t
Total mined waste volume	343 844.0	m^3
Total mined waste weigth	911 186.6	t
Cost of waste mining	**22 779 665**	**SEK**
Shotcrete thickness	0.1	m
Area of shotcrete / m	1.5	m^2
Cubic meter of shotcrete in FW1	954.6	m^3
Cubic meter of shotcrete in FW2	674.9	m^3
Cost of shotcrete	1 000.0	SEK/ m^3
Shotcrete cost in FW1	**954 560**	**SEK**
Shotcrete cost in FW2	**674 880**	**SEK**
Shotcreting performance	20.0	m^3/h
Shotcreting time in FW1	47.7	h
Shotcreting time in FW2	33.7	h
Shotcrete machine cost /h	1 000.0	SEK/h
Utility machine cost / h	500.0	SEK/h
Man-hour cost	600.0	SEK/h
Employees needed for shotcreting	2	
Shotcrete overhead & machine cost in FW1	**128 866**	**SEK**
Shotcrete overhead & machine cost in FW2	**91 109**	**SEK**

Table 9-4 Calculation table of shotcrete, mesh and bolting support method 2/2.

Number of rows in FW1	20.0	
Number of columns in FW1	52.3	
Total number of bolts in FW1	1 046.7	bolt
Cost of 1 bolt	150.0	SEK/bolt
Cost of bolts in FW1	**157 000**	**SEK**
Number of rows in FW2	20.0	
Number of columns in FW2	37.0	
Total number of bolts in FW2	740.0	
Cost of 1 bolt	150.0	SEK/bolt
Cost of bolts in FW2	**111 000**	**SEK**
Bolting performance	10.0	bolt/h
Bolting time in FW1	104.7	h
Bolting time in FW2	74.0	h
Bolting machine cost / h	1 000.0	SEK/h
Man-hour cost	600.0	SEK/h
Employees needed for bolting	1.0	
Bolting overhead & machine cost in FW1	**167 467**	**SEK**
Bolting overhead & machine cost in FW2	**118 400**	**SEK**
Bench face inclined height (meshed length)	15.2	m
FW1 surface area (mesh covered area)	9 545.6	m^2
FW2 surface area (mesh covered area)	6 748.8	m^2
TOTAL surface area	16 294.4	m^2
Mesh price per m^2	50.0	SEK/m^2
Cost of mesh in FW1	**477 280**	**SEK**
Cost of mesh in FW2	**337 440**	**SEK**
Mesh application rate	0.2	m^2/ min
Mesh application time in FW1	17.7	h
Mesh application time in FW2	12.5	h
Employees needed for mesh application	2.0	
Man-hour cost	600.0	SEK/h
Skylift for mesh application (one driving, one in the crane)	500.0	SEK/h
Mesh application overhead & machine cost in FW1	**30 051**	**SEK**
Mesh application overhead & machine cost in FW2	**21 246**	**SEK**
Cost of applied support	**3 269 298**	**SEK**
Total cost (waste mining + support)	**26 048 963**	**SEK**
Cost per meter of supported bench	**3 050**	**SEK/m**

Table 9-5 Calculation table of shotcreting and bolting support method 1/2.

Shotcrete + Bolts		
BFA	80	°
IRA	54.6	°
Bench height	15	m
Bench width	8	m
Number of benches	4	
Excavated area	1283	m^2
FW1 bench length	157	m
FW2 bench length	111	m
Waste rock density	2.65	t/m^3
Cost of waste rock mining	25	SEK/t
Total mined waste volume	343 844	m^3
Total mined waste weigth	911 187	t
Cost of waste mining	**22 779 665**	**SEK**
Shotcrete thickness	0.1	m
Area of shotcrete / m	1.52	m^2
Cubic meter of shotcrete in FW1	954.56	m^3
Cubic meter of shotcrete in FW2	674.88	m^3
Cost of shotcrete	1000	SEK/ m^3
Shotcrete cost in FW1	**954 560**	**SEK**
Shotcrete cost in FW2	**674 880**	**SEK**
Shotcreting performance	20	m^3/h
Shotcreting time in FW1	47.7	h
Shotcreting time in FW2	33.7	h
Shotcrete machine cost /h	1000	SEK/h
Utility machine cost / h	500	SEK/h
Man-hour cost	600	SEK/h
Employees needed for shotcreting	2	
Shotcrete overhead & machine cost in FW1	**128 866**	**SEK**
Shotcrete overhead & machine cost in FW2	**91 109**	**SEK**

Table 9-6 Calculation table of shotcreting and bolting support method 2/2.

Number of rows in FW1	20	
Number of columns in FW1	52	
Total number of bolts in FW1	1047	bolt
Cost of 1 bolt	150	SEK/bolt
Cost of bolts in FW1	**157 000**	**SEK**
Number of rows in FW2	20	
Number of columns in FW2	37	
Total number of bolts in FW2	740	
Cost of 1 bolt	150	SEK/bolt
Cost of bolts in FW2	**111 000**	**SEK**
Bolting performance	10	bolt/h
Bolting time in FW1	104.7	h
Bolting time in FW2	74.0	h
Bolting machine cost / h	1000	SEK/h
Man-hour cost	600	SEK/h
Employees needed for bolting	1	
Bolting overhead & machine cost in FW1	**167 467**	**SEK**
Bolting overhead & machine cost in FW2	**118 400**	**SEK**
Total cost (waste mining + support)	**25 182 946**	**SEK**
Cost of support	**2 403 281**	**SEK**
Cost per meter of supported bench	**2 242**	**SEK/m**

Table 9-7 Calculation table of meshing and bolting support method.

Mesh + Bolts		
Cost of waste mining	**22 779 665**	**SEK**
Bench face inclined height	15.2	m
FW1 surface area (mesh covered area)	9 545.6	m^2
FW2 surface area (mesh covered area)	6 748.8	m^2
TOTAL surface area	16 294.4	m^2
Mesh price per m^2	50.0	SEK/ m^2
Cost of mesh in FW1	**477 280**	**SEK**
Cost of mesh in FW2	**337 440**	**SEK**
Mesh application rate	0.2	m^2/ min
Mesh application time in FW1	17.7	h
Mesh application time in FW2	12.5	h
Employees needed for mesh application	2.0	
Man-hour cost	600.0	SEK/h
Skylift for mesh application (one driving, one in the crane)	500.0	SEK/h
Mesh application overhead & machine cost in FW1	**30 051**	**SEK**
Mesh application overhead & machine cost in FW2	**21 246**	**SEK**
Number of rows in FW1	20.0	
Number of columns in FW1	52.3	
Total number of bolts in FW1	1 046.7	bolt
Cost of 1 bolt	150.0	SEK/bolt
Cost of bolts in FW1	**157 000**	**SEK**
Number of rows in FW2	20.0	
Number of columns in FW2	37.0	
Total number of bolts in FW2	740.0	
Cost of 1 bolt	150.0	SEK/bolt
Cost of bolts in FW2	**111 000**	**SEK**
Bolting performance	10.0	bolt/h
Bolting time in FW1	104.7	h
Bolting time in FW2	74.0	h
Bolting machine cost / h	1 000.0	SEK/h
Man-hour cost	600.0	SEK/h
Employees needed for bolting	1.0	
Bolting overhead & machine cost in FW1	**167 467**	**SEK**
Bolting overhead & machine cost in FW2	**118 400**	**SEK**
Total cost (waste mining + support)	**24 199 549**	**SEK**
Cost of support	**1 419 884**	**SEK**
Cost per meter of supported bench	**1 325**	**SEK/m**

Appendix 10

Position of recommended rock mechanical boreholes in the footwall.

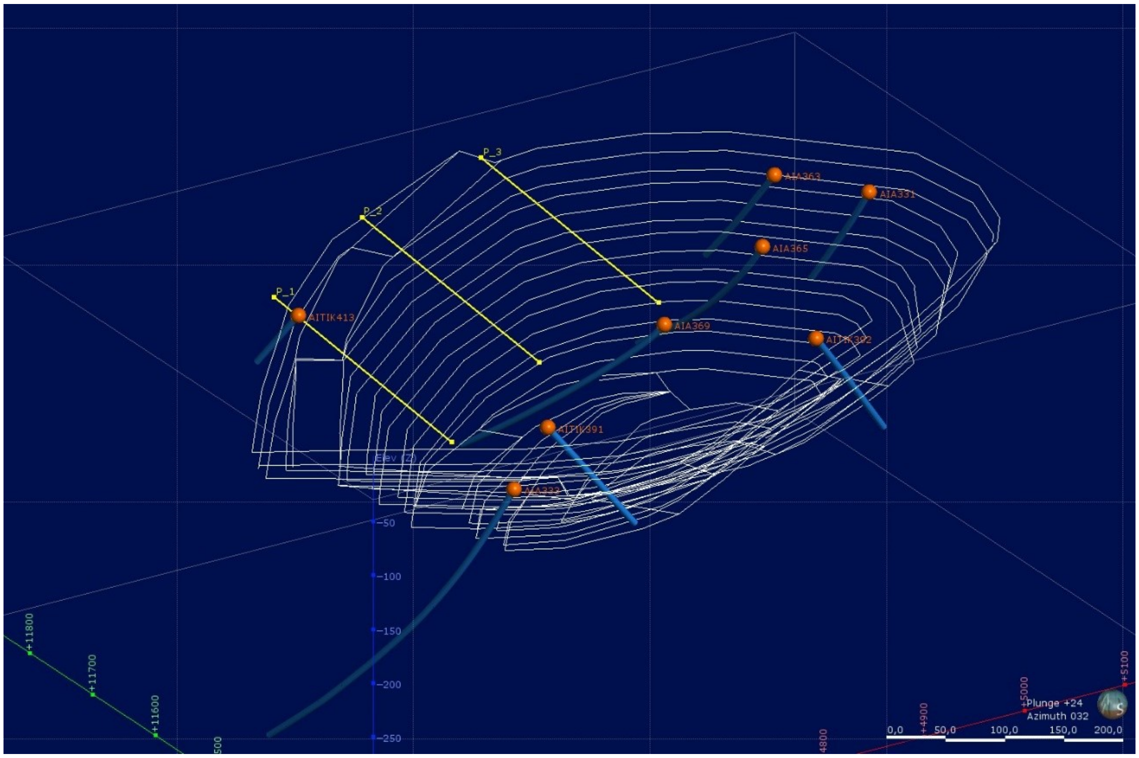


Figure 10-1 3D view of proposed boreholes. Yellow traces display the proposed drill holes, light blue traces indicate the existing holes with joint orientation information.

Table 10-1 Spatial information of the proposed rock mechanical boreholes.

	SURFACE COORDINATES		NOTE: REFERENCED TO MINE NORTH		
HOLE ID	Y [m]	X [m]	DIP [°]	DIP DIRECTION [°]	HOLE DEPTH [m]
P_1	11650	4500	45	90	250
P_2	11750	4650	45	90	250
P_3	11800	4800	45	90	250